Dike Design and Construction Guide

Best Management Practices for British Columbia

Flood Hazard Management Section
Environmental Protection Division

Province of British Columbia
Ministry of Water, Land and Air Protection

July 2003

BRITISH COLUMBIA

**Ministry of
Water, Land & Air Protection**

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Prepared by: Golder Associates Ltd. and
Associated Engineering (B.C.) Ltd.

National Library of Canada Cataloguing in Publication Data.

Main entry under title:

Dike design and construction guide: Best Management Practices for British Columbia.

Running title: Dike Guideline: Best Management Practices for British Columbia.

Includes bibliographical references: p. ISBN 0-7726-4996-0.

1. Dikes (Engineering) - Design and construction.
2. Dikes (Engineering) - British Columbia - Management.
3. Flood damage prevention - British Columbia
   - I. British Columbia. Flood Hazard Management Section.
   - II. Golder Associates.
   - III. Associated Engineering (B.C.) Ltd.

TC337.D54 2003 627.42'09711 C2003-960144-7

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DIKE DESIGN AND CONSTRUCTION GUIDE
BEST MANAGEMENT PRACTICES FOR BRITISH COLUMBIA

Many communities in British Columbia are subject to flood hazards. British Columbians need well designed and reliable flood protection works for both safety and sustained economic development. To support these objectives, this Guide consolidates and summarizes best management practices for dike design and construction, specifically for this province. It is hoped that the Guide will assist diking authorities, design professionals and others in fulfilling dike design requirements as legislated under the British Columbia *Dike Maintenance Act.*

Good dike design practice involves a variety of activities each aimed at ensuring that the new, repaired, upgraded, or changed works are constructed to an appropriate standard. To meet the standards, qualified professionals with suitable expertise must be involved. This would typically include water resources engineers and geotechnical engineers with demonstrated dike design experience. Structural design and hydraulic specialists, biologists and other professionals should also be retained to design specific project components and facilitate agency approvals.

The Guide presents design and construction standards in a generalized form only. Application of this information for specific projects requires site specific design and expert advice.

I would welcome receiving comments about the Guide and about any dike design, construction and maintenance issues in the province. Please direct these to the office of the Inspector of Dikes, or to the regional Deputy Inspectors of Dikes, Ministry of Water, Land and Air Protection.

Neil Peters, P. Eng.
Inspector of Dikes

June, 2003

105-03641/

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1.0 INTRODUCTION

Proposals for construction of new flood protection works, as well as activities on, through or adjacent to existing flood protection works, must be approved in writing by the regional Ministry of Water, Land and Air Protection (MWLAP) Deputy Inspector of Dikes (DIOD) prior to implementation (*Dike Maintenance Act*. Section 2(5)).

As part of the application and approval process for new works, or repairs or upgrades to existing works, the Inspector of Dikes (IOD) is frequently asked for information regarding basic design and/or construction considerations. For projects such as new construction and major repairs, other than routine maintenance, the DIOD will insist that the proponent procure the services of a qualified engineer. However, there is still a requirement that proponents, developers, representatives of local government and consulting professionals have a clear understanding of the principles and issues related to the design and construction of new or upgraded diking works. This document is intended to serve as a reference which summarizes current practice and illustrates design guidelines.

1.1 Purpose and Limitations of Document

Numerous criteria and issues must be considered in dike design. These may vary from project to project, and no specific step-by-step procedure covering details of a particular project can be established. However, logical steps based on successful past projects can be followed for dike design and can be used as a base for developing more specific procedures for any particular project.

This document has been developed to present basic principles used in design and construction of dikes and for the general guidance of design engineers. The guidelines convey sound engineering practices in a typical situation and detail the issues or problems which a design professional may need to resolve. This document is not intended to replace the judgment of the design engineer. The primary responsibility for proper dike design lies with the design engineer for the project.

In order for a dike to safely fulfill its intended function, the dike must also be constructed, operated and maintained properly. Supervision of construction or reconstruction of the dike by licensed professional engineers is required to ensure that the dike will be built according to the approved plans.

Other guides issued by the BC Government may assist in the proper operation, maintenance, design, and construction of dikes. To prepare an operation and maintenance manual for a dike, the "*Dike Operation and Maintenance Manual Template*" (Water Management Branch 2001) may be referenced. The "*Flood Protection Works – Inspection Guide*" (Water Management Branch 2000), "*Guidelines for Management of Flood Protection Works in British Columbia*" (Water

Management Branch 1999), and "*Flood Planning and Response Guide for British Columbia*" (Water Management Branch 1999) provide additional information related to the development and management of flood protection works. The "*Environmental Guidelines for Vegetation Management on Flood Protection Works to Protect Public Safety and the Environment*" (MELP and DFO 1999), and "*Riprap Design and Construction Guide*" (Water Management Branch 2000) may also assist in some features of dike design.

It should be noted that the term dike as used herein is defined as an embankment or structure whose primary purpose is to furnish flood protection from seasonal high water and which is therefore subject to water loading for periods of only a few days or weeks a year. Embankments that are subject to water loading for prolonged periods longer than normal flood protection requirements, or permanently, should be designed in accordance with dam criteria rather than the dike criteria given herein.

1.2 Definitions

The following list of definitions is taken from the "*Dike Maintenance Act*".

"**dike**" means an embankment, wall, fill, piling, pump, gate, floodbox, pipe, sluice, culvert, canal, ditch, drain or any other thing that is constructed, assembled or installed to prevent the flooding of land;

"**diking authority**" means:

- the commissioners of a district to which Part 2 of the Drainage, Ditch and Dike Act applies;
- a person owning or controlling a dike other than a private dike;
- a public authority designated by the Minister as having any responsibility for maintenance of a dike other than a private dike; or
- a regional district, a municipality or an improvement district.

"**improvement district**" means an improvement district within the meaning of the Local Government Act (formerly the Municipal Act);

"**inspector**" means the Inspector of Dikes(IOD) and includes the Deputy Inspector of Dikes(DIOD);

"**municipality**" means a municipality as defined for the purposes of the Local Government Act (formerly the Municipal Act);

"**order**" includes a decision or direction of the inspector;

"**private dike**" means a dike built on private property without public funds to protect only the property of a person owning the private dike.

1.3 Flooding in British Columbia

Flooding in British Columbia is caused by a variety of natural conditions which depend largely on the size and geographic location of the water body. The nature of flooding may also be

modified by natural and/or manmade changes in the watershed, as well as the presence of dams, diversions, bridges and other structures. Flood management in the province of BC falls under the jurisdiction of the Ministry of Water, Land and Air Protection (wlapwww.gov.bc.ca/wat/flood)

The Water Survey of Canada (WSC) maintains records of water levels and/or flows on major watercourses in the province and the Canadian Hydrographic Service (CHS) records sea level measurements at various locations along the coast. The appropriate records shall be referred to and analysed to appraise local conditions.

1.3.1 Flood Hazards

Freshet flooding refers to spring snowmelt runoff which is influenced by annual winter accumulation of snowpack, and specific temperature/rainfall conditions in the spring period. Snowpack accumulation/ depletion and conditions are monitored and reported on in *Provincial Snow Survey Bulletins*. Freshet flooding is capable of affecting large areas of the province. This was the case in the springs of 1894 and 1948, when widespread flooding occurred in Southern B.C.

Flash flooding may occur on small to moderately sized streams throughout the province due to spring, summer, fall or winter rainstorms. Coastal streams of all sizes commonly rise rapidly to their greatest annual peaks during intense fall and winter rainstorms and rain on snow events.

Flooding from the ocean is influenced by tides and **storm surge** which raises sea level due to barometric pressure effects and wind. A particularly sensitive period occurs if a storm coincides with spring tides. The annual tide cycle peaks around the solstices in December and June. Ocean flooding can also occur due to wave overtopping.

1.3.2 Tsunami

The outer coast, including deep fjords, is also infrequently affected by **tsunamis** or seismically generated ocean waves. Commonly referred to as **tidal waves**, these are water level changes caused by tectonic activity, and/or landsliding and may occur at any time of year. Pacific tsunami events are monitored by the International Tsunami Warning System which issues "watches" and "warnings" as required. In British Columbia, response is guided through the Provincial Emergency Program (PEP) by the British Columbia Tsunami Warning Plan. While several hours warning may be available for remotely generated tsunami, there may be little or no warning of those locally generated. Local effects will vary depending on local aspect and conditions as well as tide levels coinciding with arrival of a series of tsunami waves.

In the deep ocean they travel at high rates of speed and are difficult to observe as they are of small height. As they approach the continental shelf and shallower near shore waters, they can rapidly shoal and transform, resulting in a potentially devastating series of waves.

The B.C. coast has been subject to several tsunamis over the years, in particular the 1964 event, which was generated by a sea bottom earthquake off the Aleutian Islands of Alaska. Port Alberni was particularly hard hit by the event, and the tsunami generated high water levels in Prince Rupert and Bella Coola. Hazards similar to tsunamis include landslide generated waves. They can occur in the sea, lakes and reservoirs.

The majority of flood protection works in British Columbia are designed to protect against river and/or ocean flooding. There are currently no flood protection works in the province designed specifically to protect against tsunami occurrence.

1.3.3 Debris Flood And Debris Flow Hazards

The geography of British Columbia has resulted in the formation of a large number of **alluvial fans**, which are essentially fan-shaped deposits of water-transported material. They typically form at the base of topographic features where there is a marked break in slope. Consequently, alluvial fans tend to be coarse-grained, especially at their **apex**. At their edges, however, they can be relatively fine-grained. Fan type features can also be predominantly debris flow or debris flood produced, or related to seismic events.

Due to the terrain in the province, development has frequently occurred on fans, as they are often easier to build on than the surrounding territory. Years can go by without flooding or debris events, which in some cases has resulted in a false sense of security. Flood events can often convey water and debris, leading to potentially devastating results.

Debris flows (Photograph 1) are typically initiated during high intensity and/or prolonged precipitation events or rain-on-snow events, by various types of channel sidewall and headwall failures and, less frequently, by mobilization of streambed deposits. Estimating the actual distance a debris flow can travel is a complex process, dependant on factors such as volume and velocity of the event, the type of debris, the fan topography and the development and surface roughness of the fan area. Upon reaching the fan, where the creek channel typically loses confinement and/or steepness, the debris flow will start to deposit the transported material. Slope angles required for debris deposition to start on a fan range between 10-20 degrees from horizontal and, under some circumstances, such as coarse debris allowing for rapid drainage or large surface roughness of the fan, even at higher slope angles.

Debris floods, similar to debris flows, typically initiate during high intensity and/or prolonged precipitation events or rain-on-snow events. The general mechanism involves either the collapse of a landslide dam in the creek channel, or small landslides in the channel sidewalls. Debris floods can occur on their own or in association with a debris flow, where they form a more fluid component that can flow beyond the depositional area of the debris flow. The total amount and the size of debris

transported in a debris flood is smaller than in the case of a debris flow and, consequently, the impact forces against structures tend to be lower. Debris flood deposits are strongly controlled by topographic details and follow preferred courses such as channels, roadways and ditches.

1.3.4 Erosion Hazards

Erosion can occur as a result of sudden loss of land during a flood event, or as an ongoing incremental loss of land over an extended period of time.

1.3.5 Ice Jams and Loading

An **ice jam** is a stationary accumulation of ice that restricts flow. Upon release of the jam, constricted water can be released suddenly downstream with the potential for flooding and erosion. The presence of ice blocks or pieces in the water can exacerbate erosion due to ice impact. There are two types of jams, **freezeup jams** consisting of frazil ice that occur in winter, and **breakup jams**, made primarily of ice blocks or chunks formed by the breakup of ice cover. (Watt et al, 1989).

Photograph 1: Debris Flow, Chilliwack River Valley

Erosion can occur in the absence of significant flood events, thus for design purposes, it is necessary to consider effects due to highly erosive events, such as a design flood, as well as regular ongoing effects, such as, for example, waves from vessel traffic.

Ice loading on dikes may need to be considered during design, depending upon the climate at the proposed site. Ice impact can result in dislodgement of erosion protection due to the shear force between the ice sheets and the surface of the dike. Should the shear force be

enough to dislodge or damage the structure prior to failure or breaking of the ice sheet, loss of cover or structural damage to the dike may occur.

Failure modes against dikes would normally include bending, with crushing also occurring directly at the contact surface. Techniques are available to estimate the forces due to bending failure against sloping structures, as well as the shear forces due to ice impact (Ashton, 1986).

1.3.6 Global Warming

Global warming due to the greenhouse effect has been the subject of broad study by a number of researchers. The modeling approaches generally adopt a broadly based general circulation model (GCM), which simulate atmospheric circulation and predict changes in temperature and precipitation due to alterations in carbon dioxide concentrations. Global warming may affect, for example, the spatial and temporal distribution of precipitation with elevation, cloud cover, glacier retreat, vegetation distribution and production and plant physiology. Within British Columbia, which has a wide variation in topography, climate and geography, it is expected that global warming may affect different basins differently. Increases in temperature related to global warming may, for example, increase precipitation. However, increases in biomass within the watersheds may compensate for the higher rainfall and attenuate affects on floods. In particular, maximum annual daily flow rates may actually decrease, however, mean annual runoff may increase (Loukas et al, 1996).

Consideration for design is difficult to include on a specific basis, however, discussions with the proponent with regards to global warming shall not be overlooked in the design and approval process.

From the perspective of sea level in BC coastal areas, variations in mean sea level can arise due to seasonal changes in estuary circulation and offshore processes such as coastal upwelling, and from ocean scale adjustment to large scale weather patterns such as El Nino. Other effects may include crustal tilting and/or local subsidence. One challenge for the engineer is the fact that long term accurate records on the order of 50 to 100 years or more are required to enable identification of discrepancies between mean sea level, assuming that the tide gauge itself is not moving. Of particular note is the fact that the variation of sea level due to tidal effects on the B.C. coast is an order of magnitude greater than existing evidence of mean sea level rise.

Potential rates of sea level rise on the Fraser delta have been estimated to range between 140 and 280 mm (5.5 to 11 inches) over the next 50 years (Church, 2002).

1.4 Legislation and Regulations

1.4.1 Legislation and Regulatory Controls

Regulatory controls on construction of new flood protection works, changes and work within the existing dikes, and related maintenance are fundamentally within the purview of the BC *Dike Maintenance Act*. Legislation relevant to the formation and operation of diking authorities includes:

- Drainage Ditch and Dike Act; and
- Local Government Act.

Other pertinent provincial legislation includes:

- Emergency Management Act;
- Water Act;
- Land Act; and
- the Environmental Assessment Act

Relevant federal legislation includes the Canada Fisheries Act and the Navigable Waters Protection Act.

Construction of works and maintenance may also be subject to other municipal, provincial, and federal legislation and regulations, as well as local **bylaws** and **zoning.**

1.4.2 Dike Maintenance Act

The principal legislation in BC pertinent to flood protection works is the *Dike Maintenance Act*. Section 2(5) of the Act provides that work in and about flood protection dikes shall be subject to written approval by the IOD. This includes:

- Alterations that may lower or decrease the size and/or integrity of the cross-section of a dike;
- Installations of floodboxes, culverts, pipes or any structure in a dike;
- Construction of works over or on a dike right of way; and
- Alterations to the foreshore adjacent to a dike and excavation in proximity to the landside dike toe.

Any proposal for construction of new flood protection works, as well as activities on, through or adjacent to existing flood control works must be approved in writing by the regional DIOD prior to implementation. An application, including drawings and written description of the proposal, must be submitted for review well in advance of proposed construction.

1.4.3 Water Act

All work in and about streams or other watercourses is subject to approval or regulation under Section 9 of the *Water Act*. The Regional office of Land & Water B.C. shall be consulted as to the approval process for all projects which necessitate activity within the natural boundary of a watercourse. Fraser River Estuary Management Program (FREMP) approval for the works in the Fraser River estuary are also subject to Section 9 approval.

1.4.4 Canada Fisheries Act

The Federal Department of Fisheries and Oceans (DFO) is responsible, under the Fisheries Act (R.S.C., 1985, c.F-14), to

protect fish and fish habitat in and about "waters frequented by fish". This includes protection from any work in or near these waters. Pacific salmon are a federally-managed resource.

The MWLAP Environmental Stewardship Branch is responsible for management of steelhead, trout, char and other non-salmonid freshwater species under the Fisheries Act.

All works or vegetation removal in or adjacent to waters containing fish or fish habitat, whether marine or freshwater, require approval under the *Fisheries Act*. This will involve DFO in salmonid bearing streams and waters, and, in all cases, MWLAP Environmental Stewardship.

1.4.5 Land Act

The *BC Land Act* affects the removal of material from streambeds. Where the streambed is on Crown Land, Land & Water BC is the lead agency in terms of gravel removal, which may be subject to royalty. There may also be privately owned streambeds where the landowner must be dealt with. Note that other applicable legislation applies to both publicly and privately owned streambeds.

1.4.6 Forest Act

In all cases the Ministry of Forests must be consulted as to the approval process for removing merchantable trees and wood from streambeds and banks.

1.4.7 Canada Navigable Waters Protection Act

Works within, above or under the wetted perimeter of navigable waters will be subject to review under the *Navigable Waters Protection Act.*

1.4.8 BC Environmental Assessment Act

Major projects are subject to environmental review under the *Environmental Assessment Act.* Regulations have been developed that require review for construction of new dikes as well as raising the entire length, dismantling, or abandonment of existing structures, if protecting an area greater than ten square kilometers.

1.4.9 Local Government Act

The *Local Government Act* provides for the formation of Improvement Districts (Section 731 (2001)) for purposes designated in their Letters Patent. While there are 19 existing Improvement Districts which are now involved in diking, no new Improvement Districts will be established. The Local Government Act provides the accepted mechanisms for authorizing new diking authorities whereby municipalities and regional districts may, under local bylaw, undertake or regulate diking and drainage works within their jurisdiction.

1.4.10 Emergency Program Act

The *Emergency Program Act* regulates emergency management in the province of BC.

Section 6. (3) of the Act states that:

A local authority must establish and maintain emergency management organization to develop and implement emergency plans and other preparedness, response and recovery measures for emergencies and disaster.

1.4.11 Other Legislation

The *Drainage, Ditch and Dike Act,* passed in 1907 (consolidated 1990), provided the authority to fund and construct works for draining lands for mines, manufacturing and for municipalities or districts to be formed for that purpose. *Part 1* of the Act is no longer in use while *Part 2* is the regulatory basis for incorporation of five diking districts: Fortune Creek DD, Surrey DD, Colebrook DD, Barnston Island DD and Coquitlam DD.

1.5 Requirements for Flood Protection Works

Construction of new flood protection works or reconstruction of existing works will generally require inter-agency review and approval to meet the various legislative needs. The essential requirements for flood protection works administered under the *Dike Maintenance Act* are summarized as follows:

- Design and construction for efficient and effective operation to contain the design flood and associated forces;
- Certification of works by a suitably qualified professional Engineer;
- Provision of permanent rights of way, accesses and means of operation and maintenance; and
- Implementation by a diking authority of a continually funded dike management program including inspection, patrolling, emergency planning, operation, maintenance and repairs in accordance with an approved Operation and Maintenance (O&M) manual.

The **standard design flood** in British Columbia is the "designated flood" which means "a flood, which may occur in any given year, of such magnitude as to equal a flood having a 200 year recurrence period interval, based on a frequency analysis of unregulated historic flood records or by regional analysis where there is inadequate streamflow data available. Where the flow of a large watercourse is controlled by a major dam, the designated flood shall be set on a site specific basis." (MELP, 1999)

The criteria was originally based on the 1894 flood event which affected a broad area of southern British Columbia as the largest flood recorded in modern times. Estimates of the return period for this flood vary depending on the gauge analysed and period of record, but it falls in the range of about 1 in 160 to 1 in 200 years.

The adopted standard for the Fraser River Flood Control Program (FRFCP) was the adjusted 1894 Fraser River profile. Variations locally relate to 1948 high water marks, and adjustment of the high water profile by backwater calculation accounts for the effects of overbank flow confined by dikes. The design levels for the FRFCP sea dikes were determined by statistical analysis of coastal gauge records to determine the water level pertaining to a 0.5 % probability of annual occurrence, with addition of an allowance for waves and runup.

Elsewhere in the province, consistent with the BC Floodplain Development Control Program, the standard design flood is the "designated flood" being the flood with 0.5 % probability of annual occurrence. It should be noted that other relevant standards for older dikes and agricultural land protection works include the use of a flood of record and other criteria that represent the "standard of the day".

The design of flood protection dikes varies according to design conditions/forces, foundation conditions, and construction materials. Design forces include height and duration of high water, flow velocities, debris, seepage, internal drainage, natural processes, etc. This implies meeting a number of technical requirements, including, but not limited to:

- The profile of the design flood;
- Freeboard for hydraulic and hydrologic uncertainty;
- Landside slope stability due to steady seepage;
- Waterside slope stability due to draw down;
- Surface erosion of slopes;
- Stream erosion of the waterside slopes;
- Seepage, uplift, and piping through or under the dike and structures;
- Internal drainage;
- Permanent access for inspection, maintenance, and patrolling;
- Practicality and economy of construction and dike maintenance, and;
- Structures in and through dikes.

1.5.1 Historic Diking Standards

After the 1948 flood, the last major Fraser River flood in the Lower Mainland, regional, provincial and federal governments initiated a series of studies which lead to the establishment in 1968 of an agreement establishing the FRFCP to "undertake a program of works for flood control". The design standards for the program were developed by the Dikes, Bank protection, and Pump Work groups and approved by the Joint Program Committee. Some specific features of the standard design minimums proposed, and generally implemented, by the FRFCP include the following:

- 150 mm thick graveled crest with a minimum width of 3.66 m (12 feet);
- 3H:1V or flatter waterside sideslopes for setback dikes;
- 2H:1V or flatter sideslopes for riprap slopes ;
- 2.5H:1V or flatter landside slopes;

- 0.15 m (0.5 foot) thickness of topsoil to promote turf layer;
- Typical riprap 0.9 m (3 feet) thick over 0.3 m (1 foot) filter layer;
- Underseepage and drainage treatment as required for safety of the dike; and
- Minimum 0.6 m (2 feet) freeboard over the adjusted 1894 flood profile.

In some cases, where an existing road with a paved surface formed the dike, the freeboard was reduced to 0.3 m, as the possibility of rapid down cutting from temporary overtopping was lessened considerably (Fraser Basin Management Program, 1994).

Some of the minimum standards for agricultural dikes adopted by the FRFCP include:

- 100 mm thick graveled crest with a minimum width 3.06 m (10 feet);
- Maximum 2H:1V landside slopes; and
- Minimum 0.3 m (1 foot) freeboard over the adjusted 1894 flood profile.

In areas other than those under FRFCP jurisdiction, the standard dike freeboard for open water conditions is commonly a minimum of the higher of 0.6 m above the calculated 1 in 200 year peak mean daily flow profile or 0.3 m above the calculated 1 in 200 year instantaneous peak flow profile. Freeboard may be increased due to local conditions.

As the program was implemented over the following 25 plus years, a number of variations on the standard were constructed, due primarily to site specific issues, such as availability of material, existing old dikes and upland usage.

Current standards are discussed in Section 2.0.

1.5.2 Limitations of Dike Design in BC

Despite adherence to standards, even well engineered and constructed structures have limitations due to the nature of the design standard, uncertainty in the determination of the design conditions and forces, and ongoing changes experienced in natural systems. There are numerous limitations on dike design that affect operation and maintenance requirements, discussed as follows:

- Flood protection engineering is an inexact science that cannot completely eliminate the risk of failure. For instance, while subsurface investigation is commonly undertaken for new dikes, there is an inherent variability in natural deposits that means perfect information is seldom, if ever, available. While the engineer attempts to account for this in design practice, it is important to note that material behavior can vary along a dike and anomalies can occur. Similarly, dikes themselves are constructed largely of natural materials as engineered fills with inherent limitations on quality control.
- There are also many older dikes that have not benefited from modern

design techniques and technology which demand extra attention because of uncertainties in construction practice.

- The standard design flood in BC is similar to that used in the U.S., which has adopted the 1 in 100 year event, plus 0.9 m (3 ft). of freeboard, which equates to roughly an annual probability of 1/230 (National Research Council, 2002). On the other hand, the B.C. standard does not meet the much more stringent 0.01 % probability of annual occurrence standard for the North Sea dikes in Holland or the 0.2 % probability of annual occurrence used in parts of Saskatchewan. Not-with-standing the level of protection, it must be recognized that it is a virtual certainty that larger than design events will eventually occur.
- Other than a few major pumping facilities, dikes in BC are not designed for earthquake forces. This was necessitated by economics of treatment, although it was rationalized by the FRFCP that the chance of occurrence of a major flood peak simultaneously with a large earthquake would be rare. None-the-less, a large earthquake may cause extensive failures of saturated foundations. Resulting damage to flood protection works would need to be rapidly repaired prior to the subsequent flood period.
- Dike management contains an essential continuing component of periodic inspection, performance monitoring and assessment, and maintenance aimed at identification and correction of problems both in advance of and during large flow events. For this reason, features are routinely incorporated in dike design to facilitate the practicality and economy of O&M. For instance, dike crests are constructed to function as roads for patrol and maintenance, usually a minimum graveled width of 3.6 m with turnouts provided for maintenance vehicles. Also, in the case of the FRFCP dikes, this includes provision that grassed dike slopes be no steeper than 2.5H:1V to facilitate mowing.
- An important underlying assumption in dike design is that there is continuing post construction management including periodic inspection, performance monitoring, routine repairs and maintenance, flood patrolling as well as emergency contingency planning in anticipation of failure or larger than design events. Unfortunately, due to general economics and personnel limitations, this is not always the case. For this purpose, an O & M Manual must be prepared upon completion to provide a standard for the local authority.
- At the present time, there is no provincial standard for debris flow or debris flood hazards. The 1 in 200 year standard is inappropriate for these types of events and consideration should be given to more severe events, in the range from 1 in 500 years to 1 in 2500 years.

1.5.3 Recent and Future Construction of Flood Protection Works in BC

Environmental concerns have played a significantly larger role in design of flood protection works within the last 20 years or so. For example, since the mid 1970's, the FRFCP adopted a multi-disciplinary approach to assess environmental impacts of reconstructed works and incorporate changes preserving and protecting sensitive areas. Since that time, dike widening under the FRFCP was generally directed to the landside, rather than the streamside, to preserve waterside vegetation.

Generally, flood protection of the most troublesome areas of existing development in the province is now in place, where economic. Rehabilitation and expansion of older systems can be expected in the future, together with protection for new developments. Where new development is planned which needs flood protection, there is an opportunity for environmental concerns to be addressed at an advanced planning stage.

1.6 Consultant Selection

One of the most important procedures for ensuring proper design, construction, quality assurance and quality control, and maintenance of diking works is procuring the services of a professional engineer. The following information is designed to answer the most commonly asked questions about hiring an engineer. Further information can be obtained from the website of the Consulting Engineers of BC (www.cebc.org)

1.6.1 What Type of Engineer Should I Hire?

It is essential to select someone with a professional engineer (P.Eng.) certification, with a background in civil engineering, who is competent in the field of dike safety. Important criteria to evaluate in a prospective engineer include the following:

- A licensed professional engineer that is a current member of the Professional Engineers and Geoscientists of British Columbia;
- Experience in flood control design and construction, relative to the scope of the project;
- A knowledge of the legislation, rules and regulations governing dike design, construction and associated environmental issues in British Columbia; and,
- Specific experience in several disciplines, such as hydrology, river or stream hydraulics, structural or geotechnical engineering.

1.6.2 How Do I Choose An Engineer Who Is Best For My Needs?

There are three basic strategies for selecting engineering consulting services. These selection strategies are:

- Qualification-Based
- Fee-Based
- Intermediate

QUALIFICATION – BASED

Qualification – Based selection means that the knowledge, experience, and

ingenuity of the engineer are the determining factors in making the selection. This strategy is advantageous when the proponent is uncertain about the exact problem or the best solution to the problem.

When Qualification-Based selection is used, several engineering firms are asked to submit their technical qualifications, experience with similar projects, reputation with existing clients, and any other factors pertaining to the specific project. The proponent then selects short listed firms (generally three) to make brief presentations outlining a cost effective and innovative approach to the problem. Based upon these presentations, the proponent chooses the most qualified engineer to develop the scope of the work.

When agreement on the scope of work is achieved, the engineer and the proponent negotiate a price that is fair and reasonable to both parties. If an agreement cannot be reached, negotiations start with the second-ranked engineering firm. In this selection process, price is an important factor, but only after the most qualified engineer has been identified.

FEE – BASED

Fee-Based selection means that the engineer's fee is the only determining factor in making the selection. It is advantageous when the proponent knows exactly what is needed and can clearly define the scope of the work before meeting with an engineer. In this case, the engineer is requested to prepare the designs and bid documents or conduct investigations as the proponent specifies. This approach, while cost effective with respect to engineering fees, usually results in text book solutions, with little or no flexibility for innovative concepts.

A strict Fee–Based selection often means that the engineer selected may not be qualified to do the work, especially if the bidding is open to anyone and/or the scope of the work is poorly defined.

INTERMEDIATE

The Intermediate Option is a blended approach which draws upon certain considerations from the Qualification – Based selection and the Fee – Based selection processes. The Intermediate Option requires that the proponent pre-qualify engineers, who are then asked to submit a fee-based proposal. This process ensures a higher certainty that the work will be of superior quality, but requires the proponent to clearly define the scope of work. Without a clearly defined scope of work, the proponent may receive a wide range of fee proposals, depending on the engineer's interpretation of the project.

1.6.3 Further Considerations

Further considerations prior to selecting an engineer would include, but not be limited to:

- Requesting and contacting references from the engineer; and
- Reviewing projects that have been completed under the engineer's leadership.

It is important that the proponent maintain open lines of communication

with regulatory agencies, particularly the IOD. Careful evaluation of an engineer's recommended course of action shall be undertaken to verify that regulatory requirements will be satisfied. The proponent shall also educate themselves in the basics of flood safety and be knowledgeable regarding the laws and conditions that must be met.

2.0 DESIGN

2.1 Pre-Design Study

Prior to undertaking a diking project, a pre-design study shall be carried out and include the following components:

- Identify existing flood control works;
- Characterize the floodplain;
- Establish flood profile;
- Develop conceptual dike alignments and height;
- Identify the benefiting area of the project;
- Assess the impact of the proposed works on the environment;
- Assess the impact on existing agricultural, residential, commercial, and industrial sections within the boundaries of the flood prone area;
- Assess the impact of the proposed work on local drainage;
- Locate suitable local sources of construction materials;
- Prepare a preliminary benefit/cost assessment of the project, including enhanced property values after the project; and,
- Evaluate the hazards associated with the "do nothing" alternative.

An experienced engineer shall carry out a preliminary survey, inspect and study the area using available mapping, obtain an inventory of the existing development from the local authority, and determine the feasibility of the project.

This type of initial assessment may save cost, time, and effort required during subsequent stages of design, and the project is more likely to meet the standards required by the approving agencies.

2.2 Field Investigations

Once the dike project has been defined, whether it consists of constructing a new dike or upgrading or repairing an existing dike structure, in most cases a field investigation will be required to collect relevant information. A field investigation usually consists of an office review of all available geological, and other, pertinent information on the area of interest, an on-site survey, and subsurface investigation and testing. Some key factors affecting the extent of field investigations include:

- Construction and/or design experience in the area, particularly with respect to dikes;
- Consequences of failure involving life, property, or damage to the environment;
- Proposed final dike height;
- Expected foundation conditions (weak and compressible, highly variable along the alignment, potential underseepage and/or settlement problems);
- Borrow materials available (quality, water contents, variability); and,
- Structures in dikes and/or utility crossings.

Field investigation tasks generally include the following:

- Office study - collection and study of topographic, soil, and geological maps, aerial photographs, boring logs and well data, information and performance data on existing engineering projects, etc.
- Field survey – reconnaissance of the proposed alignment and proposed borrow areas and note observations and geology of area, documented by written notes and photographs, including such features as: riverbank and coastal slopes, rock outcrops, earth and rock cuts or fills, surface materials, poorly drained areas, evidences of instability of foundations and slopes, emerging seepage and/or soft spots, natural and man-made physiographic features, etc. Interview locals or organizations with knowledge of the foundation conditions in the area.
- Subsurface investigation – put down test holes (auger, test pits, etc.), classify materials encountered, collect samples, water table observations, possible penetration testing (DCPT's, SPT's, CPT's, etc.), possible field vane testing, possible geophysical surveys to interpolate between widely spaced test holes, etc.
- Laboratory testing – Moisture determinations, possible Atterberg limits, Gradation analyses, Consolidation tests, etc.

The extent of test holes and possible geophysical explorations is based on information such as geologic maps, airphotos, groundwater resources, prior experience in the area, and the general nature of the project.

Typical spacing of test holes usually varies from 50 to 300 m along the proposed/existing alignment, with closer spacing in expected problem areas. Test holes are normally laid out along the dike centerline with occasional test holes located near the toe of the proposed dike to provide additional information. At least one test hole shall be located at every major structure. If the dike investigation is carried out in phases, i.e. preliminary and design, additional test holes may be put down as required in the design phase of the investigation.

The depth of test holes shall be sufficient to locate and determine the extent and properties of all soil and rock strata that could affect the performance of the dike or other structures. The depth of test holes along the proposed alignment shall be at least equal to the height of proposed dike at its highest point but not less than 3 m below the existing ground surface. In the case of existing dikes, test holes put down along the centreline of the dike shall extend through the existing dike materials in addition to the above noted depths. For example, with a 3 m high existing dike, test holes put down along the centreline of the dike would extend a minimum of 6 m depth and a minimum of 3 m depth for test holes put down at the toe of the dike. Test hole depths shall always be deep enough to provide data for stability and seepage analyses of the dike and

foundation. This is especially important when the dike is located near the riverbank. Where pervious or soft materials are encountered, at least some of the test holes shall extend through the permeable material to impervious material or through the soft material to firm material. Test holes at structure locations shall extend well below invert or foundation elevations and below the zone of significant influence created by the load. The test holes must be deep enough to permit analysis of stability and underseepage conditions at the structure. In borrow areas, the depth of exploration shall extend about a metre below the practicable or allowable borrow depth or to the groundwater table. If borrow is to be obtained from below the groundwater table by dredging or other means, test holes shall be at least 3 m below the base of the proposed excavation.

Appropriate field and/or laboratory tests shall be performed in order to aid in evaluating the strength, compressibility, permeability, and erosion resistance of the foundation soils, and the existing dike materials in the case of upgrading. Also, appropriate laboratory tests shall be performed on samples of the proposed embankment materials in order to ascertain their suitability for use in the dike.

Geophysical exploration methods are a fairly inexpensive means of exploration and are very useful for correlating information between test holes which, for reasons of economy, are generally spaced at fairly wide intervals. Geophysical data must be interpreted in conjunction with borings and by qualified, experienced personnel. Since there have been significant improvements in geophysical instrumentation and interpretation techniques in recent years, more consideration should be given to their use in future dike projects.

2.3 Consultation and Approvals

All proposals for the construction of flood protection works, whether upgrading existing works or constructing new works, will be subject to municipal, provincial, and federal legislation and regulations. Project approvals and/or consultation will be required from all agencies who have an interest in the proposed project.

Approvals are generally required from the following agencies:

- Provincial Ministry of Water, Land and Air Protection - Inspector of Dikes and Environmental Stewardship;
- Department of Fisheries and Oceans Canada (DFO);
- Navigable Waters Protection Act (DFO);
- Local Municipality ;
- Land and Water BC ;
- Local river or estuary management programs; and,
- Local diking districts.

Approvals and/or consultation may also be required with:

- Ministry of Transportation and Highways;
- Local Public Transportation Agencies;
- Pipelines, utilities and other linear right of way (ROW) operators; and
- First Nations.

It would be advisable to contact all interested agencies at an early stage and discuss the proposed works. It is generally less expensive to upgrade existing dikes than to construct new dikes.

Upgrading existing works can reduce the negative impacts on the environment and adjacent water courses. It requires less additional right-of-way and less space to upgrade the existing works to the design standard, than to construct new works.

Upgrading the existing works also requires less material than constructing new dikes. The total dike settlement can also be reduced.

2.4 Design Report

The design report shall include an evaluation of the foundation conditions, the hydrologic and hydraulic design and structural stability of the proposed dike. The report shall be sufficiently detailed to accurately define the final design and proposed work as represented on the construction plans.

2.5 Construction Plans

Construction plans shall be sufficiently detailed for evaluation of the safety aspects of the dike. The cover sheet shall include a vicinity map showing the location of the works. As-built plans of the project are required upon completion of construction.

The following conventions shall be adopted for construction drawings:

- Historically, imperial stationing and chainages used 0 + 00, metric are to use 0 + 000;
- Left and right banks are established looking downstream;
- Chainages start at the downstream end and extend upstream;
- For sea dikes, chainages start at the north or west end and extend south or east.

A quantity and cost estimate is also required.

2.6 Construction Inspection

The dike's performance will largely be controlled by the care and thoroughness exercised during its construction. Unexpected subsurface conditions may be encountered which may materially affect the design of the dike. To ensure a safe design, the designer must be able to confirm design assumptions and revise the dike design if unanticipated conditions are encountered. Construction inspection is required in order to ensure that the construction work complies with the plans and specifications and meets standards of good workmanship. Therefore, construction inspection of a dike is required by a licensed professional

engineer or designate to monitor and evaluate conditions as they are disclosed and to observe material placement and workmanship as construction progresses. The engineer(s) involved in the construction of the dike work will be required to submit a periodic construction report to the proponent covering the critical inspection activities for the dike's construction/reconstruction.

2.7 Project Checklist

The proponent of a diking project shall consider the following checklist and estimate the costs of design, construction of works and the time frame required to complete the project.

.1 Selection of a Consultant (see 1-6)

- Hydrotechnical
- Structural
- Geotechnical
- Environmental

.2 Complete Project Feasibility Study

.3 Obtain Background Data

- Survey existing works or proposed alignment
- Obtain existing mapping and record drawings, identify existing property lines, rights of way
- Inventory existing development in the project area
- Locate main drainage courses
- Investigate access to site, transportation corridors and services crossing the dike

.4 Confirm Design Criteria

- Design flood level (DFL)
- Freeboard (FB)
- Design dike crest (DFL + FB)
- Flood construction elevations for residential, commercial, and industrial development
- Flood zone boundaries

.5 Complete Field Investigations

- Boreholes and test pits
- Sample analyses
- Foundation conditions
- Settlement
- Seepage control
- Erosion protection
- Environmentally sensitive areas

.6 Prepare Project Documents

- Design reports
- Project drawings
- Technical specifications

.7 Submit Project Documents to Regulatory Agencies

- Conduct preliminary discussions
- Submit necessary environmental mitigation measures and proposals
- Allow sufficient time for document review
- Submit documents for authorization

.8 Address Pre-tender Issues

- Investigate construction material sources
- Prepare cost estimates, allowing for contractor's overheads and profits

- Allow for project inspection and quality control, administrative costs & permits
- Prepare contract documents
- Complete tender call

.9 Select Contractor

- Review and compare all tenders
- Check Contractor's background and past performance
- Contact references
- Check proposed sub-contractors
- Check Contractor's equipment
- Review construction schedule and completion date

.10 Construction Supervision and Monitoring

- Review material quality
- Observe construction to ensure compliance with design, specifications and best management practices
- Process progress payments

.11 Complete Record Documents

- Prepare and certify as-built drawings
- Prepare operations and maintenance manual (see MELP, 2001 for template)
- Finalize dike right-of-way, and prepare legal plans
- Submit as constructed documents to proponent and regulatory agencies
- Include photographic records of construction activities

2.8 Civil Design Issues

2.8.1 Alignment

The **alignment** of the dike shall be selected with due regard to setback requirements, available land base for construction and site specific local constraints such as sensitive habitats. Space permitting, a setback dike has numerous benefits when compared to a waterside dike, as outlined below:

- Maintains natural wetland habitat and is environmentally sustainable;
- Provides a wider floodway with increased flow capacity;
- Reduces peak flood levels;
- Reduces flow velocity and bank erosion; and
- Reduces long-term maintenance costs due to less frequent flows against the dike slope.

Construction of waterside dikes shall be avoided unless there is an existing dike in good condition, which can be economically upgraded in an environmentally sensitive manner.

Figures 1 through 4 show various dike sections relative to the watercourse as follows:

- Figure 1 – Setback dike
- Figure 2 – Waterside dike
- Figure 3 – Overwidth dike
- Figure 4 – Overwidth dike with natural levee

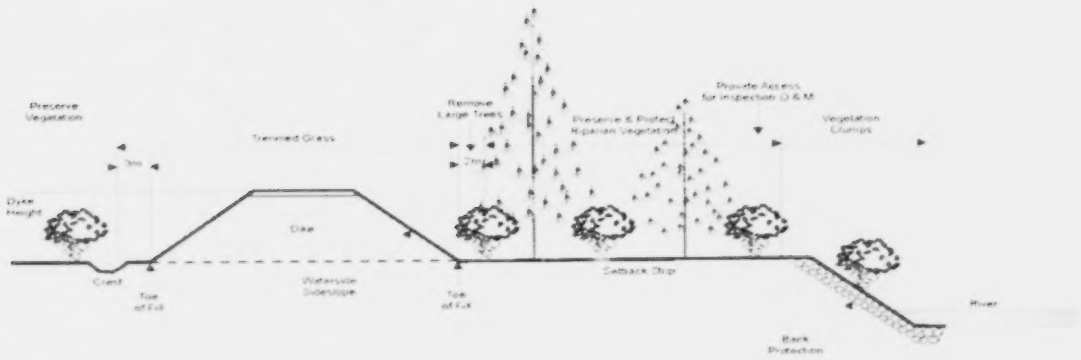

Figure 1 - Setback Dike

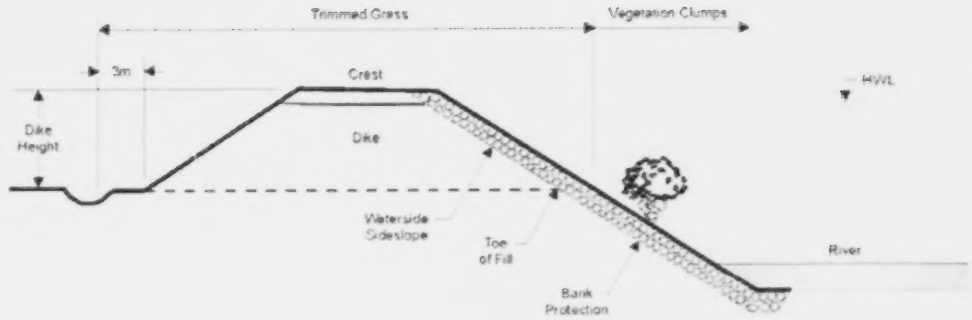

Figure 2 - Waterside Dike

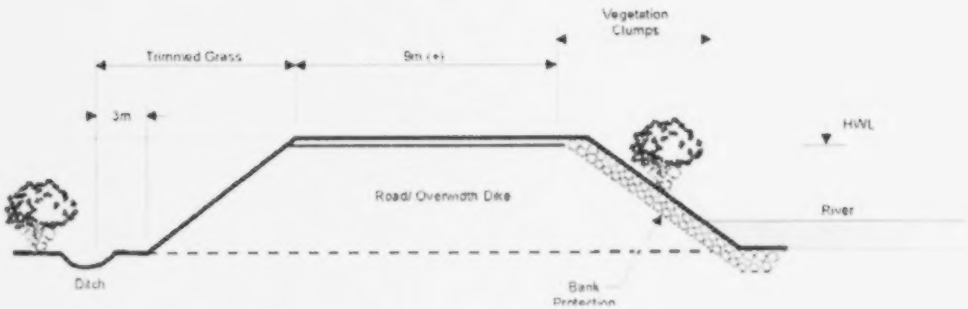

Figure 3 - Overwidth Dike

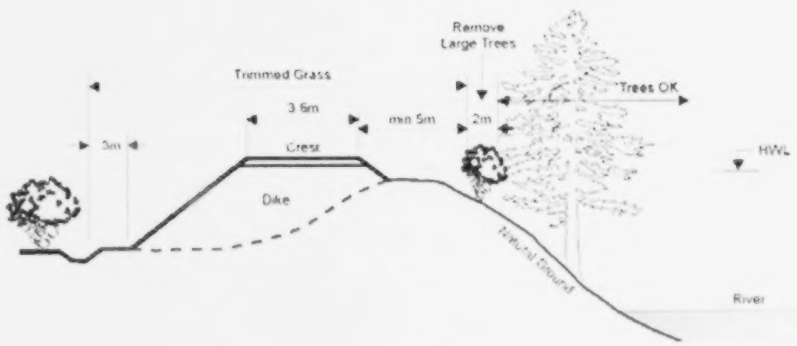

Figure 4 - Overwidth Dike With Natural Levee

2.8.2 Radius of Curvature

The dike must be configured to enable maintenance vehicles, such as trucks, a reasonable radius of curvature for safe movement, without the wheels riding over the shoulder. Therefore, consideration shall be given at the planning stage to provide manageable curves for expected maintenance vehicles.

The **radius of a curve** shall not be less than 15 m to allow efficient access of most heavy equipment. The speed at which a vehicle can round a curve is limited by the ability of the vehicle to resist centrifugal force tending to move the vehicle toward the outside of the curve. For dikes, a maximum speed of 20 km/h is appropriate (Photograph 2).

2.8.3 Flow Impingement

The issue of flow impingement is of prime importance when preparing the initial alignment of the dike. To the greatest degree practical, the dike shall parallel the direction of flow. In this manner, erosive stresses along the face of the dike during flood conditions can be minimized. By aligning the dike with the direction of flow, erosion protection requirements can be reduced.

Should the alignment of the dike be such that flow impingement during a flood event can not be avoided, erosion protection must account for flow impingement. As well, more intensive monitoring subsequent to flood events shall be undertaken. Generally, sharp bends towards the river side of the dike are not recommended.

Photograph 2: Overturned Construction Vehicle

2.8.4 Encroachment

Prior to the construction of the dike and associated flood protection works, it is essential to acquire the necessary land base, not only to build the dike but to provide additional setbacks required for environmental protection, and adequate space for inspection and maintenance of the completed works.

Temporary rights of way (ROW) may be required during the construction period for access, storage, and transportation of construction material and equipment.

For the permanent works, no encroachment on private property or environmentally sensitive areas shall be permitted, without due discussion and approval with all concerned parties. It may be necessary to pay compensation for the land base, which should be considered as part of the construction costs of the project.

A permanent ROW for maintenance purposes and future alterations shall be obtained on each side of the dike toe. The minimum width of this ROW extension shall be 7.5 m, from each dike toe, or as directed by the DIOD.

Construction on the landside of a dike shall not interfere with operation and maintenance activities. Generally, neither buildings or other permanent structures shall be allowed closer than 7.5 metres from the landside toe of a dike or as otherwise required from flood protection structures. Landside construction shall not interfere with internal drainage courses. Waterside construction shall not interfere with access for maintenance. For example, floating homes shall be setback a minimum of 7.5 m from the waterside toe of the dike.

Where permitted, fill placed against the landside slope of a dike shall be to the same standards as dike fill. Possible effects on seepage control measures shall be examined. Monitoring and/or other measures shall be provided if there is a possibility of settlement due to placement of fill.

Landscaping or recreational features, such as ponds, should not be constructed within 7.5 m of the landward side of the dike, or as directed by the DIOD.

Excavation for ponds, ditches and other features on the landward side of dikes is discouraged, as these can be problematic for access and future O & M. A ROW of 7.5 m from the toe of the dike is encouraged to control alterations and unauthorized works.

2.8.5 Viewing Platform or Lookouts

Informative tablets or plaques erected in weatherproof shelters, giving important facts and brief information on the project will be appreciated by the public, especially on large projects where a substantial amount of public funds are spent. These structures shall be removable to allow maintenance to proceed.

Information on environmentally sensitive areas and fish spawning areas can be included on these tablets. The public may have access to some dikes for walking, jogging and cycling.

Dikes can be locally widened, wherever municipal or public land is available adjacent to the dike, to provide space for benches, picnic tables and other facilities. Structures shall be located on the landward side of the dike leaving the full width of the original dike crest for maintenance and emergency vehicles.

Formal walkways and landscaped works on the dike crest can limit upgrade options, thus these are discouraged.

2.8.6 Powerlines

Power poles, towers and guy wires shall not be allowed on dikes.

Power lines shall be installed on the landward side of the dike. The power poles shall be offset a minimum of 600 mm from the landward dike toe. In this way, the overhead clearance for the lowest cable is not a concern. Power lines shall not reduce dike crest width. Where a power line crosses the dike, a minimum clearance of 5.5 m shall be provided for the lowest cable.

Power poles shall not be constructed where they would penetrate or adversely impact seepage protection works for the dike, such as toe drains.

If tall trees are present close to the power line, such trees shall be removed at the time of dike/power line construction and further growth curtailed.

2.8.7 Setbacks to Existing Dikes

Setbacks and ROWs for existing dikes shall strive to achieve the requirements set out for newly designed dikes.

2.8.8 Upgrading Existing Dikes

Investigations that shall be completed during the assessment phase for upgraded dikes include the following:

- Performance and maintenance history of existing dikes and/or flood plains;
- Flood and dike crest profiles;
- Geotechnical data related to project area; and
- Availability and quality of construction materials.

The engineer shall assess the safety and adequacy of the existing dike to confirm its suitability for achieving the required upgraded profile, stability, and seepage.

When the grade of the dike needs to be raised the preferred dike enlargement generally involves adding additional fill materials to the crest and landside slope. Other alternatives that have been used, although uncommon in B.C., are raising the dike by constructing a floodwall, "I" – type or "inverted T" – type, on the dike crown.

All low-growing vegetation and organic topsoil shall be stripped from the surface of the existing embankment before

placing new material within the area of reconstruction. The topsoil shall be stockpiled, and protected against precipitation, for later reuse.

Prior to fill placement, the stripped surfaces of the foundation and existing dike shall be scarified. As well, the existing slope shall be benched to provide an interlock between the existing and new embankment materials. The new fills shall have at least the same degree of compaction as the existing dike fills on which it is constructed.

All existing structures which are not to be operative in the proposed design, shall be removed or permanently sealed, in order to prevent a seepage path through the dike.

The raising of a dike by a small amount can result in the installation of a **sliver fill** on the landward side. This fill shall be widened to a minimum of 1.0 m horizontally to enable suitable compaction with a plate tamper or small vibratory roller. The existing fills can also be notched to enable a suitable width for compacting. Alternatively, the fill can be overbuilt and then trimmed back to a suitable dimension.

Standard practice for the construction of new dikes would include overbuilding the dike cross-section and trimming back to ensure adequate compaction.

2.8.9 Crest Width and Running Surface

Crest Width

The FRFCP recommended standard dike crest width was 3.66 m. However, the current standard crest width of 4.0 m has now been adopted. This provides easier access for construction, inspection and maintenance. In addition, if future flood profiling or recent flooding indicate a need to raise the dike, this slightly wider crest allows a nominal amount of additional material to be placed on the dike crest, while still maintaining an acceptable crest width for maintenance and inspection equipment. As well, a wider dike crest improves the support for heavy equipment. When wheels ride closer to the dike shoulders, the edges may settle and fail since the outer edges of the dike crest are unsupported.

The crest of the dike shall be sloped or cambered to promote drainage and minimize surface ponding.

Running Surface

The running surface on the dike crest will permit maintenance vehicles and construction equipment access during wet weather without causing detrimental effects such as rutting, sloughing or presenting safety hazards for inspection and maintenance personnel. A clean, well-graded, 19 mm minus sand and gravel or road mulch will provide a suitable running surface. Other alternatives which have been successfully used include 25 mm minus

crushed limestone rock and 19 mm clean crushed gravel, which is ideal over saturated dike fills. The minimum thickness of surfacing material is typically 150 mm, however, a reduced 100 mm thickness has been utilized along some dikes if the access will be generally limited to lightly loaded maintenance trucks. If the running surface is to be utilized as a higher class road, its structure is usually established by the responsible agency.

2.8.10 Dike Access

Access points are essential for emergency access to the dike during high flow periods, for routine inspections and for regular maintenance of the dike (Photograph 3).

Access roads to the dikes shall be provided at reasonably close intervals in cooperation with regulatory agencies. These roads shall be all-weather roads that will allow access for the purpose of inspection, maintenance, and flood-fighting operations.

Wherever feasible, the dike shall be connected to the local access road system. Access ramps shall be provided for approximately every two km length of dike. Ramps shall be provided at sufficient locations to permit vehicle traffic access onto and from the dike.

Access to flood protection dikes and associated structures shall not be impeded. Access roads shall be maintained, not be susceptible to internal drainage flooding during periods of high water and be wide enough to allow two lanes. A single lane with an adequate number of turnouts is also acceptable.

Where the access road is connected to the dike, the grade of the ramp shall not be steeper than 10 percent. Minimum width of the ramp shall be 4 m. The ramp shall be progressively widened toward the dike crest such that a tandem dump truck can turn in either direction of the dike.

Ramps may be located on both the landside and the waterside of the dike. The actual locations of the ramps shall have the approval of the local dike agency which owns and maintains the dike.

Waterside access ramps may be required where the riverbank erosion protection works are constructed separately from the main dike (setback dike). When used on the waterside of the dike, they shall be oriented to minimize turbulence during high water.

Parallel approach ramps shall be considered where possible, instead of right angle approach ramps, because of potential cost savings. The width of the ramp will depend upon the intended function. Some widening of the crown of the dike at its juncture with the ramp may be required to provide an adequate turning radius. Side slopes on the ramp shall not be less than 2.5H:1V to allow grass-cutting equipment to operate. The ramp shall be surfaced with suitable gravel or crushed stone. It is important to note that the dike section width shall

never be reduced to accommodate a ramp.

Turnouts and Dimensions

Turnouts are locally widened dike crest sections. These are essential during the construction of the dike as well as for regular maintenance and emergency repairs (Photograph 4).

Turnouts shall be used to provide a means for the passing of two vehicles on a single lane access road on the dike. These are especially important where access ramps are infrequent. Turnouts shall be provided within sight distance at intervals of approximately 300 m to a maximum of 500 m, provided there are no access ramps within this interval. If possible, all turnouts shall be constructed on the landward side of the dike. The exact locations of the turnouts will be dependent upon various factors such as sight distance, property lines, dike alignment, local terrain conditions and desires of local interests. An example turnout for a dike with a 4 m dike crown is shown in Figure 5 (top diagram).

Turnouts shall be evenly spaced between access ramps. In the completed dike, there will be turnout points for trucks at approximately one kilometre intervals, or less.

Minimum dimensions that have been used successfully for typical turnouts are as follows:

- The turnout shall have an extra 6 m width over and above the 4 m wide dike crest.
- The extra width shall extend for approximately 20 m length, with 15 m taper sections on both sides,

Photograph 3: Access Roads For Dike Maintenance

resulting in a total length of the turnout of 50 m.

A suitable running surface, similar to the dike crest shall be placed on the turnout.

Turnarounds

Generally dikes are terminated connecting to existing high ground. Road access does not normally exist at these points, thus vehicle turnarounds shall be constructed. Turnarounds are also essential when the dike access is cutoff by railway or highway embankments that are higher than the dike crest. The high ground can be cut back to build the turnaround or fill can be placed, preferably on the landside. A suitable running surface shall be installed on the turnarounds similar to the dike crest.

Turnarounds shall be provided to allow vehicles to reverse their direction on dikes where:

- dead-ends exist;
- dike is longer than a kilometer; and
- no access ramp exists in the vicinity of the dead-end.

The turnaround shall also allow two trucks to pass one another with one parked on the turnaround, and the other travelling on the normal crest width. An example turnaround for a dike with a 4 m crest is shown in Figure 5 (bottom diagram).

Photograph 4: Access Ramp and Turnout

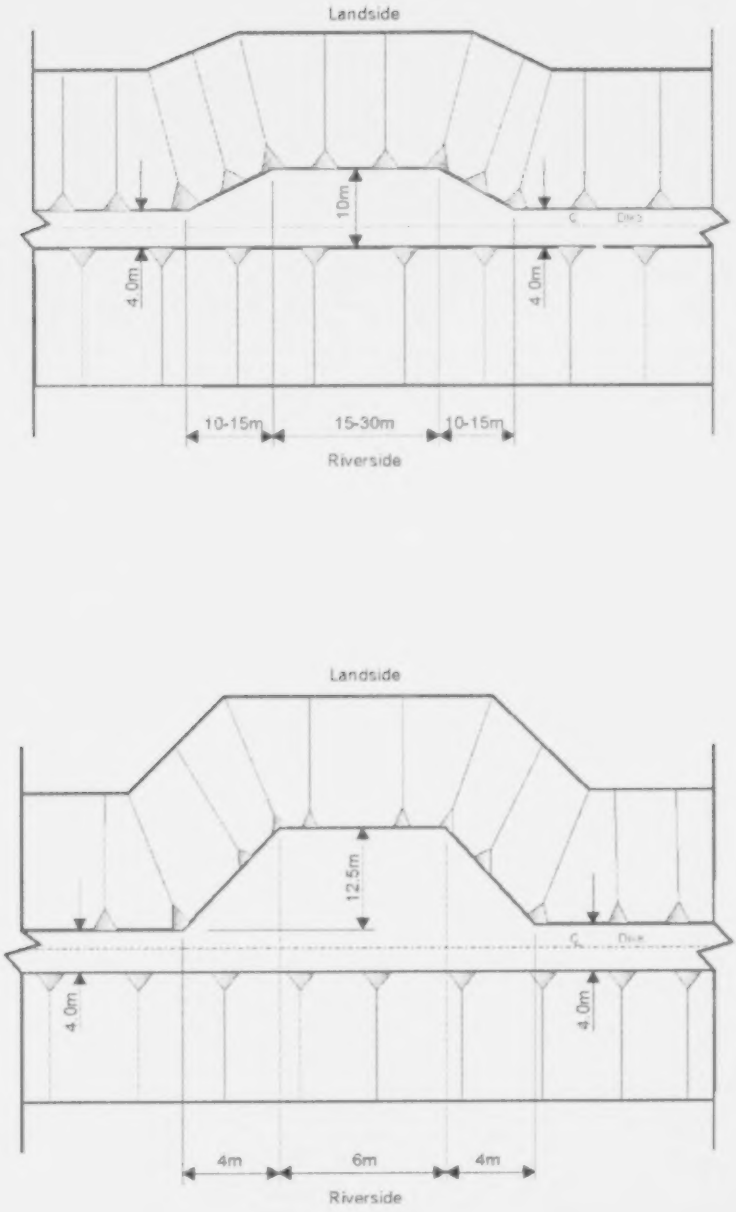

Figure 5 - Turnouts and Turnarounds

2.8.11 Dredging Limits

Dredging of the channel fronting the dike shall not take place in any location that would tend to undermine or threaten the stability of the dike or bank protection.

Specifications for dredging adjacent to dikes are site specific, however, dredging shall not be undertaken within 10 metres of the toe of the riprap, nor extend below a 3H:1V slope extending from the riverward shoulder of the dike crest.

2.9 Design Criteria

The site-specific details that shall be considered in the design of dikes are:

- foundation conditions;
- dike stability with respect to shear strength;
- settlement, seepage, and erosion;
- available dike materials;
- available construction equipment; and
- available area for ROW.

Proposed cross-section designs shall be analysed for stability as it is affected by foundation and/or embankment shear strength, settlement caused by compression of the foundation and/or the embankment, external (surface) erosion, and internal erosion (piping). The methods described and referenced herein contain procedures that have proven satisfactory from past use.

In the case of the FRFCP program, dikes were designed to be stable under conditions of design water levels for prolonged periods of time. The dike slopes varied with each site depending on the individual situation of soils, dike height, and dike construction materials. Generally the dike slopes were 3H:1V, or flatter. In many cases, a landside toe drain was required to control seepage pressures. In some cases, uplift pressures on weak soil layers had to be controlled by relief wells drilled through the weak layer. These wells relieve the pressure by allowing controlled flow. Alternatively, uplift was controlled by berms placed at the landside dike toe which counteract the uplift force. Where required, ditches previously excavated at the landside toe as a source of dike material, were filled to increase stability and prevent piping.

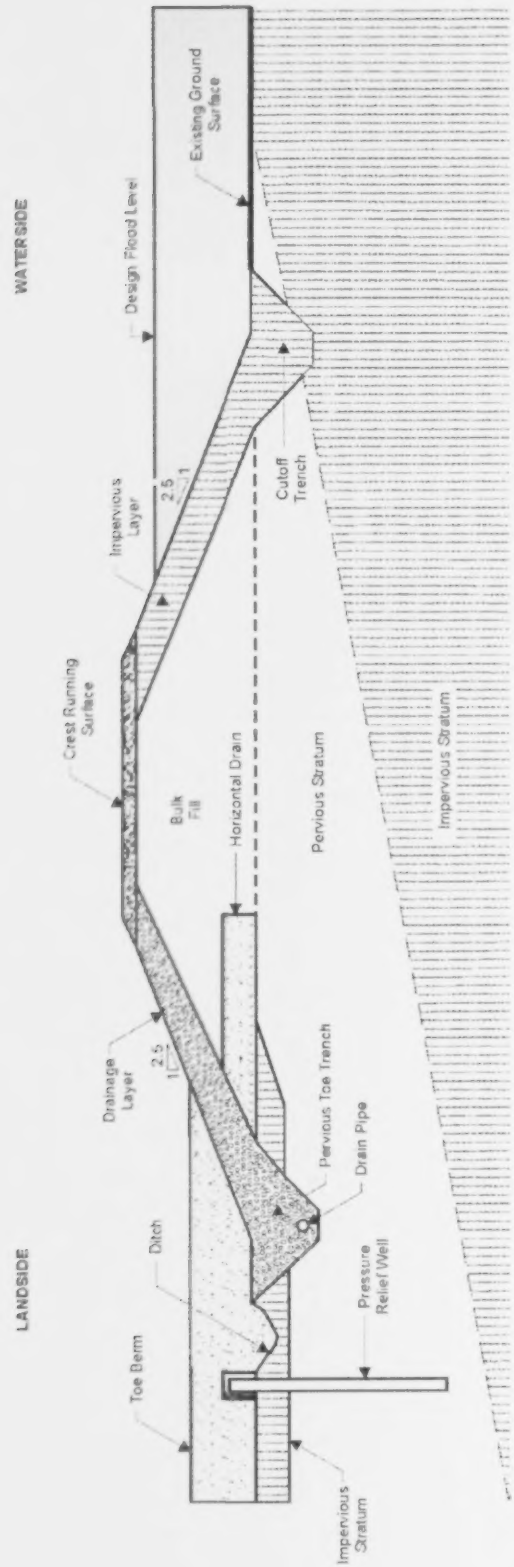

Figure 6 - Potential Seepage Control Options for Design (Not All Required)

2.9.1 Seepage

Seepage is the movement of water through soil under a differential hydrostatic pressure and can result in problems with dike stability. When carrying out a new dike design or upgrading an existing dike, the potential for and expected problems related to foundation underseepage and seepage through the dike shall be seriously considered. Issues related to seepage are discussed herein with each of the seepage methods depicted on Figure 6.

Foundation Underseepage

Without proper control, underseepage in pervious foundations beneath dikes may potentially lead to sand boils (Photograph 5), piping beneath the dike, and/or result in excessive hydrostatic pressures beneath a relatively thin impervious top stratum on the landside of the dike. Underseepage problems are common where a pervious substratum underlies a dike and extends both landward and waterward of the dike or where a relatively thin impervious top stratum exists on the landside of the dike, underlain by pervious materials. Underseepage problems shall be assessed for both new dikes and upgrades to existing dikes, many of which have not been designed for adequate underseepage control. The most common seepage control measures to reduce or eliminate foundation underseepage include cutoff trenches, waterside impervious layers, landside seepage berms, pervious toe trenches, and pressure relief wells. These methods will be discussed in the following sections.

Photograph 5: Sand Boils

Photograph 6: Cutoff Trench

Cutoff Trenches

A **cutoff** beneath a dike to reduce seepage through pervious foundation strata is typically the best means of reducing underseepage problems. Cutoffs are typically located at the waterside toe of the dike and generally consist of excavated trenches backfilled with compacted impervious fill. Cutoff trenches are generally only effective when they penetrate through the pervious strata, or at least 95 per cent into it, and will rarely be economical where they must penetrate more than a few metres depth. However, if a dike is less than 3 m in total height, some reduction in underseepage can be achieved with a cutoff trench which extends to a minimum 1 m depth (Photograph 6).

Open cut trenches can be excavated above the groundwater table, but a dewatering system will likely be required if the trenches need to extend below the groundwater table. This is due to the likelihood of the sideslopes of the trench sloughing below the groundwater table. Dewatering dike foundations for the purpose of excavation and backfilling in the dry is expensive if more than simple ditches and sumps are required, and shall be avoided if possible. To avoid dewatering, the sides of the open cut trench may be supported with sodium bentonite clay and water slurry. These slurry materials can then be displaced with a relatively impermeable fill material.

Consideration could also be given to driving steel sheet piles along the toe of

Photograph 7: Construction of a Landside Seepage Berm

the waterside of the dike to reduce underseepage. While this method is not entirely watertight due to leakage at the interlocks, it can reduce the possibility of piping in the case of sandy strata in the foundation.

In BC, cutoff trenches have been utilized for a number of dike structures and generally extend to less than 2 m depth. Deeper cutoff trenches, slurry trenches, and sheet piling have been less common in the construction of dikes in BC.

Waterside Impervious Layers

Dikes in BC are frequently founded on natural covers of relatively fine-grained impervious soils overlying pervious materials. If these natural covers are continuous and extend waterward for a considerable distance, they can effectively reduce seepage flow and seepage pressures on the landside of the dike. Where underseepage is a problem, waterside excavation shall be avoided if possible. Consideration may also be given to thickening the relatively impervious cover by placing impervious materials in areas where it thins out, depending on the effectiveness of this cover based on analysis.

Landside Seepage Berms

If uplift pressures in pervious deposits underlying an impervious top stratum on the landside of a dike become greater than the effective weight of the top stratum, heaving and rupturing of the top stratum may occur, resulting in sand boils or blow outs. Where space is available, the construction of **landside berms** can reduce or eliminate this hazard by providing the additional weight needed to counteract these upward seepage forces and the additional length required to reduce uplift pressures at the toe of the berm to acceptable levels. Landside berms also reduce the potential for sloughing or failure of the landside slope. Seepage

berms may be placed on pervious foundations or reinforce a relatively thin existing impervious top stratum. Other advantages of these berms are their relative ease to construct and limited maintenance requirements (Photograph 7).

The type of seepage berm used shall be based on available fill materials, space available on the landside of the dike, and relative costs. The most common types of seepage berms are impervious berms and sand berms. Relatively impervious berms need to be constructed to the thickness necessary to provide an adequate factor of safety against uplift from underlying seepage pressures and sand berms shall be as pervious as possible to allow excess pore pressures to dissipate, thereby reducing the potential of piping and sloughing at the landside dike toe. Sand berms typically require less material and space than relatively impervious berms providing the same protection. These landside berms shall fall within the landside dike ROW. Landside seepage berms have been used along the Matsqui dike and the Abbotsford side of the Vedder canal.

Pervious Toe Trench

Where a dike is founded on pervious materials overlain by little or no impervious materials, a partially penetrating **toe trench** (Photograph 8) can improve seepage conditions at or near the toe by reducing the potential for a buildup of seepage pressure. The main use of a pervious toe trench is to manage shallow underseepage and protect the area in the vicinity of the dike toe. A perforated pipe can be placed at the base of the trench to collect the seepage. They may also be used in conjunction with relief well systems where the wells collect deeper seepage and the trench collects shallow seepage.

Pervious drainage trenches are typically located at the landside dike toe, but are occasionally constructed beneath the landside dike slope and connected to a horizontal pervious drainage layer located at the base of the dike and as discussed below. Trench geometry will depend on the volume of expected underseepage, desired reduction in uplift pressure, construction practicalities, and the stability of the material in which it is being excavated. The trench is generally excavated to a minimum 1 m depth for dikes which are less than 3 m in height.

Photograph 8
Installation of Pervious Toe Trench

Photograph 9: Pressure Relief Wells

The granular backfill for trenches must be designed as a filter material in accordance with criteria given in Section 2.9.6. If a collector pipe is used, the pipe shall be surrounded by a 300 mm thickness of gravel and may need to be surrounded by a geotextile fabric to limit the migration of fine grained materials into the gravel or the collector pipe.

Pressure Relief Wells

Pressure relief wells (Photographs 9, 10) are used where pervious strata underlying a dike are too deep or too thick to be penetrated by cutoffs or toe drains or where space for landside berms is limited. They may be installed along the landside toe of dikes to reduce uplift pressure which may otherwise cause sand boils and piping of foundation materials. Wells accomplish this by intercepting and providing controlled outlets for seepage that would otherwise emerge uncontrolled landward of the dike. Relief wells shall adequately penetrate pervious strata and be spaced sufficiently close to intercept enough seepage to reduce to safe values the hydrostatic pressures acting beyond and between the wells. Relief well systems can be easily expanded if the initial installation does not provide the control needed and the discharge of existing wells can be increased by pumping if the need arises. A relief well system requires minimal additional land compared with other seepage control methods but also require periodic maintenance and frequently suffer loss in efficiency with time, due to clogging of well screens. The increase in seepage discharge and means for collecting and disposing of their discharge must be provided. If relief wells are to be used in the design, the diking authority must be informed of and be prepared to accept the additional maintenance responsibilities.

The design of a pressure relief well system involves determination of well spacing, size, and penetration to reduce

Photograph 10: Installation of Pressure Relief Wells

uplift between wells to allowable values. Factors to be considered are:

- depth, stratification, and permeability of foundation soils;
- distance to the effective source of seepage;
- characteristics of the landside top stratum, if any, and;
- degree of pressure relief desired.

Proper methods of drilling, backfilling, and developing a relief well must be employed or the well will be of little or no use.

In BC, pressure relief wells have been used along the Chilliwack side of the Vedder Canal where a permeable sand layer is overlain by a relatively thin weak silt layer. The wells relieve the hydrostatic pressure created against the silt layer to reduce the chance of piping or blowouts. It should be noted however, that pressure relief wells are uncommon in BC due to their relatively high cost.

Seepage Through Embankments

During high water conditions, most dikes have a certain amount of seepage resulting from water percolating slowly through the dike and its foundation. Seepage occurs in all dikes and is considered normal as long as the water remains clear and the flow is not concentrated or changes dramatically in velocity and quantity. However, it is generally better to be proactive and reduce the probability for potential seepage problems by designing a dike, new or upgraded, with some seepage control, where appropriate.

Should seepage through an embankment emerge on the landside slope it can soften fine grained fill in the vicinity of the landside toe, causing sloughing of the slope, or even lead to piping or

Photograph 11: Flattening Sideslopes to Install Impervious Layer

internal erosion of any fine sand and silt materials which may exist. It could also result in high seepage forces, decreasing the stability of the slope. In many cases, high water stages do not act against the dike long enough for this to happen. If analysis indicates that potential problems may result from the emergence of seepage from the slope, provisions shall be incorporated in the dike section such as impervious layers, pervious drainage layers (or toe), or horizontal toe drains to prevent or reduce seepage from emerging on the landside slope. For the most effective control of through seepage, the permeability shall increase progressively from the core out toward the landslide slope, depending on the materials available.

Impervious Layer

If a relatively pervious material is used for the general dike construction and limited seepage through the dike is preferable, an impervious layer is commonly placed at the waterside of the dike to eliminate, or greatly reduce the amount of seepage passing through the dike. This impervious layer is typically 0.5 to 1.0 m in thickness and extends across the entire waterside slope of the dike. An impervious layer can also be combined with cutoff trenches, which have previously been discussed, as a method for controlling underseepage (Photograph 11).

Drainage Layer or Toe Drain

A **pervious drainage layer** or **toe drain** will provide an exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. A pervious drainage layer (or toe) can also be combined with partially penetrating toe trenches, which have

Photograph 12: Drainage Layer and Pressure Relief Wells

previously been discussed, as a method for controlling shallow underseepage. A pervious drainage layer is typically 0.5 to 1 m in thickness and extends across the entire landside slope of the dike. The dimensions of a toe drain are dependent on the geometry of the dike. If the difference in gradation between the bulk dike fills and the pervious drainage materials are great, the drainage layer may need to be separated from the dike fills with a graded filter or geotextile fabric to reduce the potential for the migration of soil from the dike core material into the generally coarser grained drainage layer, or toe (Photograph 12).

Horizontal Drainage Layers

Horizontal drainage layers essentially serve the same purpose as a pervious drainage layer or toe drain but can extend further under the embankment and typically require less material. They can also serve to protect the base of the embankment against high uplift pressures where shallow foundation underseepage is occurring. These layers typically have a minimum thickness of 500 mm.

In BC, horizontal drainage layers have not been commonly used but shall be considered for new dike construction.

Design of Drainage Layers

The design of pervious drainage layers or toe drain and horizontal drainage layers must ensure that such layers have adequate thickness and permeability to transmit seepage without any appreciable head loss while at the same time preventing migration of finer soil particles. The design of drainage layers

must satisfy the criteria outlined in Section 2.9.6 for filter design.

Methods of Analysis

If the thickness of an existing surficial impervious layer, underlain by pervious soils, is greater than the expected reservoir head, it can generally be assumed that there will be no major problems involved so far as underseepage or seepage forces are concerned. In most other cases involving new dikes or upgrades to existing dikes, both underseepage and through seepage shall be assessed. This may involve simple analyses in the case of low dikes, less than 1.5 m in height, or more detailed analyses for higher dikes. In many cases, if a designer has had significant experience with local dikes, seepage control measures may be based on past experience and may not require extensive analyses. Also, in some cases seepage control measures may be avoided by flattening the slopes of the dike enough to reduce seepage problems by increasing the seepage path under a dike.

To estimate the amount of underseepage that may be expected, it is necessary to determine the coefficient of permeability of the pervious foundation. To determine a reasonable approximation of these coefficients, the values may be estimated using the results of gradation analyses carried out on samples of the pervious materials, or pump tests where water is pumped into drill holes and the rate of seepage is observed under a given head. When more precise estimates of the coefficient of permeability are required for analyses, methods such as pump out tests, rate of travel of a dye or electrolyte from the point of injection to an observation well, or a number of laboratory tests, have been used successfully in the past.

Various methods of analysis are available for the number of possible underseepage scenarios as determined appropriate by the designer. Examples of these include the Darcy formula (amount of underseepage for simple analyses), and using a flow net (seepage forces for more complex analyses). Computer programs are also available which greatly reduce the effort of making such analyses, and primary attention can then be devoted to the more important problems of defining the coefficient of permeability. One such program that is commonly used for simple and complex seepage analyses is Seep/W (Geo-Slope International Ltd., 2002), which uses the finite element method of analyses. This program may also be used in conjunction with Slope/W (Geo-Slope International Ltd., 2002) to analyse seepage and stability problems together.

2.9.2 Fill Settlement

Like uncontrolled seepage, settlement of a dike can result in failure of the dike, but more likely will serve to precipitate failure by another mode such as seepage or shear failure. Consolidation, shrinkage, and some lateral deformation occur over a period of time. Some lateral deformation can occur quickly,

however, particularly during construction. Settlement problems are almost always related to fine-grained soils, such as silts or clays. Settlement and/or shrinkage of coarse-grained soils, such as sands and gravel, is generally much less than for fine-grained soils and occurs quickly, usually during construction.

Settlement of dikes can result from consolidation of foundation and/or embankment materials, shrinkage of embankment materials, or lateral spreading of the foundation. Excessive uniform settlement can cause a loss of dike height. Differential settlement can result in cracking of the dike, which can then lead to a shear or piping failure. This is an especially acute problem at the contact between a dike and an adjacent structure.

Settlement of the dike and any other related structures shall be evaluated and provisions made in the design to counteract the effects of any anticipated settlements which in many cases involves overbuilding the dike. Where ongoing settlement is expected, provisions shall be made to monitor the dike crest elevation for settlement for up to a two year period with the dike topped up periodically as settlement occurs, or to verify settlement if overbuilding was carried out. If monitored settlement persists, a longer monitoring period than two years maybe required.

Settlement Analyses

Settlement estimates can be made by the design engineer using standard analysis methods. Detailed settlement analyses shall be made when significant consolidation is expected, as under high embankment loads, embankments of highly compressible soil, embankments on compressible foundations, and beneath steel and concrete structures in dike systems founded on compressible soils. Where foundation and embankment soils are relatively pervious, most of the settlement will occur during construction. Where analyses indicate that more foundation settlement would occur than can be tolerated, partial or complete removal of compressible foundation material may be necessary from both stability and settlement viewpoints. When the depth of excavation required to accomplish this is too great for economical construction, other methods of control such as staged construction or vertical sand drains may have to be employed, although they seldom are justified for this purpose.

2.9.3 Fill Slope Stability

Overstressing of low shear strength soils in the dike and/or the foundation, often coupled with seepage effects, is the cause of most dike failures. Failures of this type can be the most catastrophic of all since they usually occur quickly and can result in the loss of an entire section of the dike. These failures may involve the dike alone, or they may involve both the dike and the foundation.

For dikes constructed in BC on generally competent foundation conditions it has been found that dike side slopes are typically stable at slopes of 2H:1V, or flatter. The FRFCP standard for dike construction was 3H:1V, or flatter, for waterside sideslopes without riprap protection, 2H:1V, or flatter, with riprap protection, and 2.5H:1V, or flatter, for landside slopes, to facilitate mowing.

For dikes of significant height, greater than 2 m, or when there is concern about the adequacy of available embankment materials or foundation conditions, dike embankment design requires detailed analysis. Low dikes and dikes to be built of good and well compacted material which rest on proven foundations, may not require extensive stability analysis. For these cases, practical considerations such as type and ease of construction, maintenance, seepage and slope protection criteria control the selection of dike slopes. In these cases, as a general rule, landside slopes without seepage control measures shall be no steeper than 3H:1V, and shall be no steeper than 2H:1V, with adequate seepage control. For the waterside slope, the slopes shall be no steeper than 2.5H:1V, and may be steepened to 2H:1V, with erosion control. It shall be noted that dikes with heights of greater than 2 m and less than 3.5 m shall be sloped at 3H:1V, or flatter, even under the most favorable conditions. Higher dikes require further analyses.

In the case of upgrading an existing dike, the slope stability shall be checked for the existing condition and the expected condition following upgrading. Placement of stockpiles, heavy equipment, or other surcharges may also cause instability and shall be analysed.

Methods of Analysis

Analyses of slope stability involves three basic parts:

- obtaining subsurface information;
- determining soil strength parameters; and
- determining a potential failure surface which provides the minimum safety factor against failure for various water level stages.

The principal methods used to analyse dike embankments for stability against shear failure assume either a sliding surface having the shape of a circular arc within the foundation and/or the embankment, i.e. slip-circle analyses, or a composite failure surface composed of a long horizontal plane in a relatively weak foundation or thin foundation stratum connecting with diagonal plane surfaces up through the foundation and embankment to the ground surface, i.e. wedge analyses. Various methods of analysis are available for each of these scenarios and can be chosen for use where determined appropriate by the designer. Some methods commonly used to carry out these types of analyses include the Bishop, Janbu, and Spencer methods. Computer programs are also available which greatly reduce the effort of making such analyses, and primary attention can then be devoted to the more important problems of defining the

shear strengths, unit weights, geometry, and limits of possible sliding surfaces. One such program that is commonly used for simple and complex stability analyses is Slope/W (Geo-Slope International Ltd., 2002).

Conditions Requiring Analysis

The various loading conditions to which a dike and its foundation may be subjected and which shall be considered in analyses include end of construction, sudden drawdown from full flood stage, and steady seepage from full flood stage, which represents fully developed phreatic surface. The **phreatic surface** or zero pressure surface is the upper surface of seepage and is referred to as the phreatic line in cross section. Although the soil may be saturated by capillary above this line, giving rise to a line of saturation, seepage is limited to the portion below the phreatic line.

Another type of analyses that may be considered, but that is not required at this time in BC, are the effects of earthquake, or seismic loads on dike stability.

The steady-state condition that involves the maximum saturation of the embankment is the most critical post construction condition for the stability of the landside slope. The most critical operating condition so far as the stability of the waterside slope is concerned is a rapid drawdown after a long period of high reservoir level.

End of construction

The analysis is based on undrained conditions for impervious embankment and foundation soils, i.e. excess pore water pressures exist due to insufficient time for the soil to drain following placement/loading. For most pervious soil conditions, it can be assumed that they drain fast enough during placement/loading so that no excess pore water pressure exists at the end of construction. This condition applies to both the waterside and landside slopes.

Sudden drawdown

Analysis is based on the condition where a prolonged flood stage saturates at least the major part of the waterside embankment portion and then falls faster than the soil can drain. This condition only applies to the waterside slope.

Steady seepage from full flood stage (fully developed phreatic surface)

Analysis is based on the water remaining at or near the full flood stage level long enough to fully saturate the embankment. This condition may cause a condition of steady seepage which may result in the landside slope becoming unstable. This condition only applies to the landside slope.

If the factor of safety against slope stability failure is considered to be too low, methods commonly used to improve embankment stability by changes in the embankment geometry include flattening embankment slopes and the addition of stability berms. Methods to improve stability on dikes being constructed on weak or

compressible foundations are discussed in the following section.

Methods of Improving Stability

Dikes proposed on foundation soils that cannot support the dike embankment because of inadequate shear strength require some type of foundation treatment if the dike is to be built. Foundation deposits that are prone to cause problems broadly include very soft clays, sensitive clays, native organic deposits, debris deposited by man, and loose sands. Very soft clays are susceptible to shear failure, failure by spreading, and excessive settlement. Sometimes soft clay deposits have a zone of stronger clay at the surface, caused by desiccation, which if strong enough may reduce the need for treatment. Sensitive clays are brittle and even though possessing considerable strength in the undisturbed state, are subject to partial or complete loss of strength upon disturbance. Most organic soils are very compressible and exhibit low shear strength. The behavior of debris deposited by man, such as industrial and urban refuse, is so varied in character that its physical behavior is difficult, if not impossible, to predict. Loose sands are sensitive to disturbance and can liquefy and flow when subjected to shock or even shear strains caused by erosion at the toe of slopes. However, failure of loose sands is mostly earthquake related and not presently required for analyses in BC.

The most effective method of dealing with excessively compressible and/or weak foundation soils which are not excessively deep, is to remove them and backfill the excavation with suitable compacted material. Other options to deal with these types of soils include displacing the materials by end dumping new fills onto them (also useful for access road construction), and staged construction.

Staged Construction

Staged construction refers to the building of a dike over several time intervals (i.e. in stages). This method is used where the strength of the foundation material is inadequate to support the entire weight of the embankment if constructed at a pace that does not allow the foundation materials to drain. Using this method, the embankment is built to intermediate grades and allowed to rest for a time before placing more fill. Such periods permit dissipation of pore water pressures, which results in a gain in strength so that higher embankment loadings may be supported. Initial estimates of the time required for the needed strength gain can be made from results of consolidation tests and study of boring data. Piezometers shall be installed during construction to monitor the rate of pore water dissipation, and rate of fill placement shall be based on these observations, together with direct observations of fill and foundation behavior. Disadvantages of this method are the delays in construction operation, and uncertainty as to its scheduling and efficiency.

If the expected rate of consolidation under staged construction is unacceptably slow, it may be increased by the use of prefabricated vertical or wick drains. Such drains are geotextile wrapped plastic cores that provide open flow areas in the compressible stratum. Their purpose is to reduce the length of drainage paths, thus speeding up primary consolidation. The wick drains are very thin and about 100 mm wide. They can be pushed into place through soft soils to over 30 m depth. Before the drains are installed, a sand drainage blanket is placed on the foundation which serves not only to tie the drains together and provide an exit for escaping pore water, but as a working platform as well. This drainage blanket shall not continue across the entire base width of the embankment, but shall be interrupted beneath the center.

The layer above the drains must be cutoff as a flow path under the dike. An upstream clay blanket should be used. The clay should be placed after settlement is complete. A toe drain should be included to manage seepage.

2.9.4 Dike Materials

A wide range of materials may be considered for use in dikes depending on seepage and stability considerations, and materials available for use within the vicinity of the proposed dike alignment (or upgrading). If a fine-grained soil can be brought readily within the range of water contents suitable for compaction and for operation of construction equipment, it can normally be used for embankment construction. Generally, any dike fills proposed as bulk fills, or specific impervious or pervious fill layers shall limit the particle size to generally less than 100 mm and shall not contain materials greater than 150 mm in diameter. Larger cobbles and boulders within a dike fill make adequate compaction difficult to achieve and may allow void space to remain in the fills following compaction.

To limit seepage through a dike, the bulk dike fill would typically comprise between about 15 and 30 per cent fines (silts and clays) passing the U.S. Standard No. 200 sieve. For increased resistance to seepage, a material with greater than 30 per cent fines may be considered but will likely be more difficult to compact unless it is near its optimum moisture content. This type of material can be relatively impervious when well compacted. If most of the readily available borrow area materials contain less than 10 per cent fines, they may be considered for use as bulk fill if an impervious layer is placed on the waterside of the dike. The fill materials for an impervious layer shall be a clay based material with greater than about 25 percent fines with limited oversize particles (i.e. 75 mm minus).

Any proposed pervious drainage layers placed on the landside of the dike shall contain less than 5 percent fines passing the U.S. Standard No. 200 sieve. If this material is not available in the vicinity of the dike and needs to be imported, a 75 mm minus well-graded sand and gravel fill shall be considered.

A careful analysis of all available material sources, including location, material type, and available volume shall be made (Photograph 13). At least 15 m in width shall be left undisturbed between the toe of the dike slope and the edge of the borrow pit.

Generally, proposed borrow materials shall have natural water contents low enough to allow placement and adequate compaction. The cost of drying borrow material to suitable water contents can be very high, in many cases exceeding the cost of longer haul distances to obtain material that can be placed without drying. Borrow soils undergo seasonal water content variations; hence water content data shall be based on samples obtained from borrow areas in that season of the year when dike construction is planned. Possible variation of water contents during the construction season shall also be considered.

In computing required fill quantities, a shrinkage factor of at least 25 percent shall be applied. For example, borrow area volumes shall be at least 125 percent of the dike cross-section volume. This will allow for material shrinkage, compaction, hauling and other losses.

Borrow areas shall be cleared and grubbed to the extent needed to obtain fill material free of objectionable matter, such as trees, brush, vegetation, stumps, and roots. Topsoil with low vegetative cover may be stripped and stockpiled for later placement on landside slopes of dikes and any additional berms.

Photograph 13: Grizzly For Production of Dike Materials

2.9.5 Earthquake Considerations

Other than a few major pumping facilities, dikes and dike structures in BC are not designed for earthquake forces. This was necessitated by economics of treatment, although it was rationalized by the FRFCP that the chance of occurrence of a major flood peak simultaneously with a large earthquake would be rare. However, substantial deformation of dikes in tidal areas shall be given special consideration.

It shall be noted that if foundations or the dike materials consist of saturated, low relative density (i.e. loose) sands and silts, or uniform, cohesionless materials are encountered, serious damage may result to the structure during a moderately large earthquake due to liquefaction of those materials. Liquefaction of a saturated soil results from disturbance of the grain to grain contact and the consequent transfer of the load to the water in the voids in the soil. This leads to a rapid build up of water pressure in the soil pores, during which time the soil acts as a liquid. Some possible effects of liquefaction at a site include loss of bearing capacity, settlement, the potential for lateral spreading, especially near river or sea slopes, and embankment failures.

In BC, the potential for liquefaction pertains particularly to the Vedder Canal, Richmond, and Delta dikes, many of which are composed of fine, loose sand and silt founded on a thin silty layer which are underlain by a thick layer of generally loose to compact fine sands. Resulting damage to flood protection works would need to be rapidly repaired prior to the subsequent flood period, or prior to high tides.

While it may not be possible to justify earthquake design, studies shall be carried out to determine the expected locations and extent of damage to specific dikes for different return periods, the time required to rebuild following an earthquake, and the return period for which dikes and foundations are stable. Of particular concern is whether it would be possible to rebuild the threatened dikes before the next freshet.

Riprap placed steeper than 2H:1V may be potentially unstable during a seismic event. The consideration of final slope configuration will include the cost savings of placing less material at a steeper slope, and shall also consider reconstruction costs in a major seismic event. For significant structural components, such as pump stations, consideration of seismic issues shall be included in the evaluation of construction cost and overall project risk. If components survive a seismic event with little damage, repair costs will be less.

2.9.6 Erosion Protection

Riprap Layer

The riprap layer is the primary protection against shear stress or erosive forces from flowing water which can act to remove material from the face of the

dike. In BC, rock riprap is normally the most cost effective erosion protection material, due to its durability, history of use and availability. Riprap erosion protection (Photograph 14) is normally easy to repair, straightforward to construct and can withstand some dislocation of the armour without failing. Limitations include the slope angle, which has a minimum standard slope of 2H:1V, difficulty in placing underwater to a suitable finish tolerance, environmental acceptability as well as aesthetic appearance.

Erosion protection is constructed to withstand the following specific hydraulic processes that can cause erosion of dikes:

- The action of water flow, including frictional erosion due to shear stress, direct impingement flow, eddying due to restrictions in the channel;
- The action of debris floods and debris flows, which can include all of the aforementioned scenarios;
- Wave action resulting in wave breaking and overtopping; and
- Ice impact.

Design criteria for riprap erosion protection include the following:

- Density and strength criteria of rock;
- Angular character of rock;
- Durability under freeze thaw;
- Soundness and texture;
- Required size distribution to withstand the estimated design forces;
- Layer thickness and placement technique;
- Use of toe aprons or berms to prevent undermining; and
- Final placed slope angle.

Design guidelines are presented in the

Photograph 14: Rock Riprap Erosion Protection

manual entitled "Riprap Design & Construction Guide" (Water Management Branch, 2000).

Filter Layer

Filter layers are defined as layers to protect the underlying dike core material from erosion by currents or waves without excessive pore water pressure buildup in the core. The filter can consist of one or more layers of granular materials, geotextile fabrics or a combination of fabric overlain with granular material.

Filter layers are designed to achieve the following objectives:

- prevent migration of underlying finer grained soils into the overlying erosion protection layer; and
- reduce hydrodynamic loads on the dike's outer stone layers, by allowing additional dissipation of flow energy.

Granular filters can have the following advantages over geotextile filters in dike construction:

- the filter elements are usually very durable;
- granular filters provide a good bedding layer between the erosion protection layer and the underlying core;
- self weight of the filter layer contributes to its stability during construction, while geotextiles may need to be anchored; and
- the nature of the granular filter allows it to better withstand in parts when the erosion layer is being placed.

However, granular filters can be difficult to place underwater, as uniform construction thicknesses are not easy to achieve.

Design criteria for granular filters include the following:

- retention criteria to prevent loss of the foundation or core material;
- permeability criteria to reduce hydraulic gradients across the layer;
- internal stability criterion to prevent loss of the finer fraction of the filter layers itself; and
- layer thickness.

If a geotextile filler is used, it is recommended that a bedding layer be placed over the geotextiles to prevent damage from stone impact.

Riprap Tie-in (or Key-in)

Adequate tie-in or tie-back is required to protect against out flanking by floodwaters. The "Riprap Design and Construction Guide" (Water Management Branch, 2000) provides direction on design and construction of tie-ins.

Groynes

Groynes protect erodible banks by directing the flow toward the centre of the channel. They are useful in reducing meander migration and water velocities

near the bank. The design variables most used for deflector design are:

- orientation angle and effective length;
- spacing of groynes;
- placement site construction material; and
- adequate tie-in to the bank is required to protect against outflanking.

2.9.7 Flood Levels – River Dikes

In British Columbia, the standard design flood is the flood with the annual probability of occurrence of 0.5 %, also referred to as the 1 in 200 year flood, on the local reach of the river or the stream where the flood protection works are built. The standard design flood levels for rivers in British Columbia shall be determined by the proponent, in consultation with the Flood Hazard Management section of the Ministry of Water, Land and Air Protection.

The standard for river dike crest elevation is the higher of 1 in 200 year instantaneous flow plus 0.3 m freeboard, or the 1 in 200 year maximum daily flow plus 0.6 m freeboard. For agricultural land, the higher of the 1 in 50 year instantaneous flow plus 0.3 m freeboard or the 1 in 50 year maximum daily flow plus 0.6 m freeboard is the recommended minimum level.

2.9.8 Flood Levels – Sea Dikes

For dikes bordering the ocean, the dike crest height is estimated considering the following contributions:

- tidal fluctuations;
- storm surge; and
- wave runup.

The maximum high tide can be derived from the CHS Tide Tables for the BC Coast, either by inferring directly from stations adjacent to the study area, or by interpolating from one of the reference stations.

Storm surge occurs as a result of barometric pressure variation and other atmospheric effects. The storm surge magnitude can be estimated from a comparison of actual measured water levels and predicted water levels at a tide reference station. Annual maxima can be analysed statistically to estimate, for example, a 1:200 year storm surge. One then normally includes the storm surge, plus high tide, plus a freeboard (normally 0.6 m), to establish the 1:200 year water level for a coastal site.

Additional considerations for dike height on a site specific basis may include **wave runup** and **setup**. Wave runup is the limit of wave uprush on the seaward face of the dike, and wave setup is the superelevation of the water line landward of the breaker line. In most instances, wave runup is an order of magnitude larger than wave setup, and can be estimated for the depth limited wave height at the toe of the structure.

2.9.9 Freeboard

The profile of the dike crest is obtained by adding a freeboard allowance to the standard design flood. **Freeboard** is the vertical allowance added to the standard design flood levels to allow for uncertainties in flood levels. The standard design flood profile is discussed in Section 2.9.7.

Additional freeboard may be required where channel infilling, aggradation, debris accumulation, or log jams may occur. As well, the freeboard may be increased to allow for long term dike settlement.

Reference information on design flood levels for many rivers in B.C. can be obtained from the Flood Hazard Management Section of MWLAP.

2.9.10 Design Discharge Estimation

Design discharge estimation is the process by which a particular flow rate is determined for the watercourse in question. There are two approaches whereby the design discharge can be estimated and the procedure depends upon whether the watercourse is gauged or not.

Gauged System

Should the watercourse be gauged, flood flows corresponding to various return periods can be estimated using a statistical analysis of the gauged data. Several statistical distributions are available and the selected distribution will be chosen based upon goodness of fit, consistency of instantaneous and daily flood estimates and regional suitability. The proximity of the gauge to the study area shall be carefully examined and the quality and length of the dataset confirmed. The estimated flow rates at the gauge location can then be adjusted using an areal adjustment procedure to incorporate differences in drainage basin sizes from the recording site to the site of interest. Further considerations may include the effect of other tributaries and timing of peak flows in those tributaries with respect to the main system.

Ungauged System

To estimate design discharges in an ungauged system, one can draw upon gauged data sets in adjoining similar watersheds. These gauged watersheds shall also be reviewed for proximity to the site, and similarity in physical characteristics, for example, watershed size, hypsometry, exposure and hydrologic zone to determine their suitability for data transfer and adjustment.

Flows corresponding to various return periods are then estimated statistically, and flows for the target watershed estimated based upon areal adjustment. Alternatively, if the areas of adjoining watersheds lie above and below the area of the target watershed, the flood flows for specific return periods can be interpolated directly or on a water yield per unit area basis.

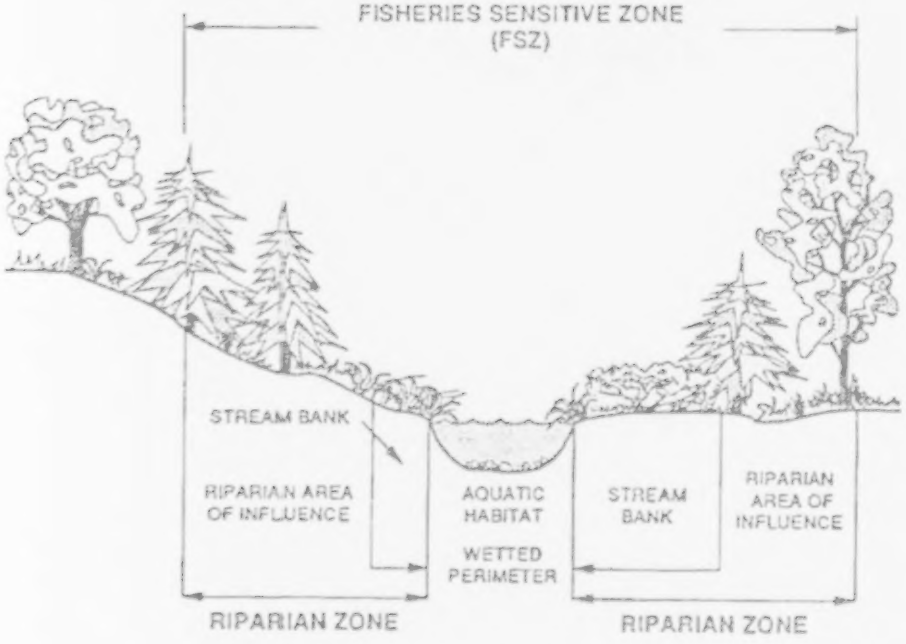

Figure 7 – Fisheries Sensitive Zone

There may also be occasions when there are no suitable local gauges for transfer to the target watershed. In these instances, flood flows can be estimated using a runoff routing approach whereby a synthesized design rainfall event can be used to estimate runoff and hence flood flows. Difficulty with this technique can arise if there are no data or observations on past floods to enable some measure of calibration to occur.

2.10 Environmental Issues

2.10.1 Mitigation requirements

Construction of dikes will generally lead to the implementation of mitigation works, such as plantings or habitat features, in order to offset disturbance of existing habitats or vegetation. The mitigation requirements for a project will be determined by the relevant environmental agency, such as DFO, and generally will require extensive consultation in the design phase of the project prior to settling upon a final alignment and configuration.

2.10.2 Fisheries Sensitive Zones

Fisheries sensitive zones (FSZ) are defined as zones adjacent to channels or watercourses which have the potential to support fish. FSZ are classified under the Canadian Fisheries Act and provincially under the Environmental Stewardship Branch of MWLAP. In the absence of specific habitat maps, such as those prepared for the Lower Fraser by the Fraser River Estuary Management Program (FREMP), the FSZ generally

includes the watercourse from top of bank to top of bank, and extends landward a distance not less than 30 m. Figure 7 details a typical FSZ designation.

Every opportunity shall be taken to minimize encroachment into the FSZ, recognizing that if this does occur, mitigation of habitat loss may be required.

Floodplain areas have meandering streams and marsh habitat. The streamside vegetation and the aquatic insects that breed and reproduce in the wetland habitat along the stream banks contribute to the food chain. The trees and the riparian vegetation along the banks provide the shade to keep the water cool during the summer and regulate the water temperature during the winter. Therefore, most floodplain areas are fisheries sensitive zones.

Construction of a diking project may alter the natural habitat and have significant detrimental effects on the fish habitat. Careful planning and implementing of habitat mitigation, compensation and environmental enhancement measures, within or locally outside the proposed flood protection area, may achieve both flood protection and environmental protection objectives.

When a stream is intercepted by the dike, pump station and floodbox structures are required to transfer the internal drainage flow to the outside of the dike. These structures also interfere with the normal migration of fish to spawning and rearing grounds located inside the diked area. Construction of fish friendly structures such as screw pumps and floodboxes with horizontally opening floodgates can minimize fish mortality.

In bank protection works, **approved vegetation** can be incorporated within the rock riprap layer to provide some degree of natural vegetation. Acceptable guidelines for vegetation are detailed in the "Guideline for Vegetation Management" (MELP, 1999).

2.11 Structural Issues

2.11.1 Buoyancy of Structures

Wherever possible, buoyant structures shall not be placed in the dike. However, placement of structures subject to uplift forces in the dike is sometimes unavoidable, and is often required as part of the flood control strategy. Structures such as pump stations, flood boxes, gas pipelines, and partially empty sewer lines may be subject to uplift forces. Flotation can be avoided by selecting heavier construction materials, providing base slabs that extend beyond the structure walls, and by tying down the structure using anchors, seepage/cutoff collars and/or headwalls.

Buoyant uplift could also occur in gas pipelines and partially empty sewer lines, as well as other enclosed chambers within the body of the dike. Counter weights and anchors can be adopted to prevent this possibility.

2.11.2 Pump Stations

Pump stations are an essential feature of the flood protection works. Pump stations (Photograph 15, 16) will discharge the internal drainage across the dike to the main river, when the floodboxes are closed by the high tide or high river elevations.

The operation of the pumps can be minimized by providing sufficient internal storage facilities such as wide drainage canals and storage lagoons. However, the use of pump stations can rarely be eliminated.

Pump stations are generally built with floodboxes as combined structures. Combined structures can achieve considerable savings in excavation, temporary cofferdams, de-watering, materials used for construction such as formwork, concrete and reinforcement. Also, considerable savings can be achieved for inlet headwalls, wing walls, trash racks, screens, outlet works and erosion protection works which are all essential items. When the structures are built separately, each structure requires these components for satisfactory operation and maintenance.

In the design of pump stations, the following essential features must be considered:

- A reliable power supply. Pumps generally require 600 volt – 3 phase power supply;
- The inlet and the pump chambers must conform to the Hydraulic Institute Standards;
- The pump or a combination of pumps must have adequate capacity to handle design storms;
- The pump station building must be on a stable foundation. Preloading of the site or piling may be required. Site investigation and advice by a geotechnical engineer is recommended;
- The building design shall permit easy installation and removal of

Photograph 15: Pump Station

pumps for future maintenance and repairs;

- Consideration shall be given for the installation of fish friendly screw pumps;
- The control room and the electrical controls must be housed above the internal flood level;
- Adequate heating, ventilation and lighting shall be provided for the control room;
- Air vents shall be provided for the pump discharge chambers;
- Debris control and collection facilities to prevent debris entering the pump intake by the installation of trash racks and debris deflecting log booms. These can also prevent beaver activity and potential blockage of inlets;
- Stable forebay area, not subject to deposition or erosion. Wingwalls, headwalls and rock rip-rap filters may be required at the inlet;
- Forebay area shall provide sufficient storage volume to prevent frequent starts and stops of the pumps, which can be harmful for the pump motors;
- Discharge area designed to prevent erosion. Erosion protection is required for discharge flow pipes. Trash racks or screens are also essential at the outlet to prevent floating debris and log deposition against floodgates;
- Safety features such as handrails and barriers must be installed at inlet and outlet;
- The pump control room must have steel doors. Pump chamber covers and access covers must be steel secured with locks to prevent vandalism and damage to equipment;
- Staff gauges shall be installed at both inlet and outlet to monitor internal and external water levels. This will help for future maintenance and keeping records of high water levels.

Photograph 16: Pump Station

- Staff gauges shall be tied to Geodetic Datum;
- Standby generators or emergency power supply equipment may be considered;
- Also portable pumps complete with portable generators may be required during extreme high water levels and flooding periods;
- Alarms and remote monitoring system to detect pump failure and high water levels. Radio or telephone equipment must be installed;
- Provide flood lighting at inlet and outlet areas for emergency inspections during night and to deter vandalism;
- Copy of Pump Station Operation and Maintenance Manual relating to the installation must be kept at the station; and
- Operations log must be maintained and kept at the station.

2.11.3 Floodboxes

A **floodbox** is a drainage culvert through the dike that conveys the internal drainage from a watercourse from inside the dike to the main river outside the dike. A gate is installed at the outlet end of the floodbox to prevent back flow from the river to the inside and to allow gravity flow from inside to the outside. This can occur only when the outside water level is lower than the inside water level. The gate must have a proper seal to prevent leakage at high river or high tide periods. Trash racks shall be provided at the inlet and outlet to prevent debris from becoming lodged in the gate.

Depending on the volume of flow, a floodbox may consist of a single culvert or a series of culverts. At each flood box installation, at least one side mounted gate shall be provided to allow larger openings during smaller differential heads. This is beneficial for fish passage.

The normal top-mounted flapgate design was changed in some installations to a side-mounted design part of the way through the FRFCP program at the request of DFO to provide easier fish passage.

Cast-in place & precast concrete box sections can be used for the construction of single-bay or multi-bay floodboxes. Use of precast concrete sections can reduce construction time, dewatering costs, formwork, reinforcement installation, concrete supply and placement and other on-site costs. However, cast-in-place seepage cutoff collars may have to be built on site.

The use of corrugated steel pipes (CSP) as floodboxes across dikes is not recommended. Drainage flows contaminated by fertilizer and flow coming through peat bog areas or brackish water causes the CSP to corrode.

CSP has been used for non-acidic flows, but must be zinc or asphalt coated to prevent corrosion in acidic flows.

Photograph 17: Headwall With Wingwalls and Flapgate

Alternative methods include circular concrete and high density polyethylene (HDPE) pipes. The issue of long term maintenance shall be discussed between the engineer and the proponent. Items that have a cheaper capital cost may require more maintenance in the future. Any selected materials shall be approved by the IOD.

Proper design of joints and connections for pre-cast concrete sections, particularly where settlement of the structure may occur, is required.

2.11.4 Headwalls and Wingwalls

Headwalls and wingwalls (Photograph 17) prevent erosion and sloughing of dike slopes at floodbox and pump station inlet and outlet areas. Headwalls and wingwalls help reduce the length of the floodboxes and pump intake and discharge structures.

Headwalls can also act as seepage cutoff walls at structures, but shall not be included in the total seepage collar calculation.

2.11.5 Floodgates

Floodgates are generally fabricated from plate steel and structural sections. Gates consist of two parts, a frame and a gate. The frame complete with a neoprene seal around its perimeter is attached or cast onto the outside face of the floodbox. The gate is made as a separate unit with hinges and is attached to the frame. All components shall be galvanized with zinc coating.

Manufactured gates are available up to a certain size. Side mounted gates are not readily available, especially for larger openings. Manufactured gates are normally used for small round floodboxes.

In agricultural areas where there is a need to feed back irrigation water from

the river to the interior, a sluice gate is installed. An irrigation intake tunnel can be built with the floodboxes or the sluice gate can be installed on the pump discharge outlet to allow backflow from the river to the interior.

2.11.6 Guard Rails and Hand Rails

Safety handrails and guardrails can be securely installed on headwalls and wingwalls. Guardrails and handrails shall be provided in accordance with Worker Compensation Board (WCB) requirements and local building codes. Vehicle barriers shall be provided where damage to dike, floodbox, pump station or other flood protection works could occur.

2.11.7 Dike Construction Adjacent to Structures

When a dike is abutted against a concrete or other structure, items that shall be considered in the design of the abutment include differential settlement, compaction, and embankment slope protection.

Differential settlement caused by unequal consolidation of the foundation soil at the abutment between a dike embankment and a typically lighter concrete or other structure can be serious if foundation conditions are poor and the abutment is not well designed. Preloading has been used successfully in the past to minimize these potential differential settlements.

Proper compaction of the dike embankment at the abutment of the

Photograph 18: Compaction of Dike Materials Adjacent to Structures

structure and dike is essential (Photograph 18). Good compaction decreases the permeability of the embankment material and ensures a better contact with the structure. Heavy compaction equipment shall be used as close to the structure as possible and smaller equipment such as plate tampers shall be used immediately adjacent to the structures and in any other confined areas.

Slope protection and/or flattening of the dike sidelopes shall be considered where dikes abut concrete or other structures due to the potential for turbulence to occur at the juncture of these structures and the possible increased chance of instability.

2.11.8 Bridges and Stream Structures

Approvals by the IOD for such structures will be limited to assessing the impact of the proposed crossing on existing dikes. If the proposed bridge crosses an existing dike the clearance between the dike crest and the soffit of the bridge must permit trucks to pass over the dike. This clearance shall not be less than 4.5 m over the dike crest.

In addition to conventional engineering design practice regarding construction of bridges, the following shall apply to bridges and similar structures in diked areas:

- structures (Photograph 19) shall not present an unreasonable obstruction to the floodway, or the channel of the watercourse and floodplain required to pass the design flow;
- the hydraulic effects of the bridge, including probable debris and ice jams, shall not increase the dike design profile upstream of the bridge;
- bridges and related works shall be situated and constructed to minimize

Photograph 19: Bridge Clearance

Photograph 20: Floodwall

opportunity for blockage, diversion, redirection or change in velocity of flows, scour, sedimentation and / or interference with access for operation and maintenance of the flood protection works;

- spans are preferred to be clear and not alter the natural channel width;
- piers, abutments and in-stream support structures shall be oriented and shaped to convey flows efficiently parallel to the natural direction of flow without aggravating or causing unnatural impingement on the flood protection works or presenting propensity for blockage;
- abutments and piers shall be founded on piles or piers extended below river scour levels and protected against erosion and scour. Bank protection shall be aligned and oriented to provide hydraulically smooth banks and tied in to existing flood protection works;
- backwater effects, head losses, changes in flow direction or velocity shall not be caused at the design flood level;
- the underside of a bridge shall have a minimum clearance of 1.5 m above the higher of the calculated peak instantaneous 1:200 year level or flood level of record, or higher as required for ice or debris passage;
- If demonstrated to be uneconomic or not feasible, clearance requirements may be varied if there is an acceptable debris/ice management program in effect or determination of acceptable risk of blockage. Under no circumstances will the underside of a bridge be lower than the higher of 0.3 m above the peak instantaneous 1 in 200 year level or 0.6 m above the 1 in 200 year mean daily flow.

Foot bridges shall be designed to the same guidelines described in Bridges and Stream Structures.

2.11.9 Floodwalls and Retaining Walls

Retaining walls and floodwalls (Photograph 20) are built where physical

Photograph 21: Lock Block Retaining Wall with Gangway to Float

restrictions and space limitations do not allow the construction of a standard earthfill dike with acceptable slopes.

Retaining walls are also built to overcome obstructions along the dike alignment. The presence of existing buildings and other structures in close proximity to the dike toe may prevent the construction of a sloped embankment. Short length retaining walls are built to save partial demolition or removal of the existing structures.

Lock block walls are a particular configuration of retaining wall which makes use of individual concrete units for the wall. Construction is rapid, as the units come precast with keys and keyways. A suitable foundation layer of granular crush is required for the base, with suitable backfill and drains being required. Seepage between individual blocks must be prevented; grouting is one approach which has been well accepted.

2.11.10 Marinas/Docks/Boat Ramps

Marine structures such as marinas and docks, or debris booms, shall be designed to minimize opportunity for flow deflection, trapping of debris or ice, or other effects that may affect the stability or safety of the dike. Consideration of the access for dike maintenance is also required, in particular under or adjacent to docks and ramps. A further issue to note may be the provision of utility service corridors, such as lights and power, through the dike to enable lighting and so forth on the marina (Photograph 21).

Provision should be made to prevent erosion by boat wash, or wakes. This may include, for example, altered riprap

or erosion protection works, as well as operational criteria such as vessel velocity, in close proximity to the dike.

Also, boat ramps must not jeopardize the integrity of the dike.

2.11.11 Seepage Reduction Around Pipes and Culverts

Seepage tends to creep along the relatively smooth surface of pipes, culverts and floodboxes placed within the dike fill. There are a few effective methods for reducing this seepage. In British Columbia, seepage collars or cutoff walls have been historically used. The difference between these devices is that a number of seepage collars are typically used along a pipe, while typically only one cutoff wall is used along a pipe or other structure.

Photograph 22:
Seepage Collars

Seepage collars and cutoff walls help minimize the probability of flow paths along the soil-pipe interface. In British Columbia, increasing the seepage path by 20% using these devices has been traditionally used. Seepage collars or cutoff walls are required on all pipes or structures that extend through a dike. This requirement can be waived for pipes 50 mm or smaller that are located in the freeboard allowance of the dike. Seepage collars should be built around the outside perimeter of the pipe projecting at least 600 mm beyond the edge of the pipe. The seepage collars should be spaced at maximum 6 m centres along the pipe centreline. They should be located at least one metre from pipe joints, and if possible, midway between pipe joints. Where possible, cutoff walls should be located within the impervious core of the dike and sufficiently far from the embankment surface to prevent a reduced seepage path.

Consideration may be given to designing seepage collars or cutoff walls by utilizing the Lanes' weighted creep method (Lane, 1935, Rijkswaterstaat, 1999). The collars/cutoffs would be designed such that Cw, the seepage factor, is increased to a conservative value.

Seepage collars (Photographs 22, 23 & 24) can be built using concrete, steel plate or high density polyethylene sheet. Whatever the material used, it is important to provide a proper seal along

Photograph 23: Cutoff Wall Installation

the contact face of the pipe. In case of steel or polyethylene pipes, steel or polyethylene walls can be continuously welded or bolted onto the pipe. Concrete walls can be built around steel and concrete pipes to provide the seal.

In this case, there should be a neoprene or similar seal between the collar and the

Photograph 24: Cutoff Wall Installation

Photograph 25: Transverse Pipe Crossing

pipe. Cast-in-place concrete surrounding the pipe is also acceptable.

Piping failure can occur where seepage collars or cutoff walls are used if the material around the seepage collar/cutoff wall has been poorly compacted at its contact with the structure. Adequate compaction is critical, in particular around the lower half of the collar, or under the haunch of the pipe.

An alternative that has become more popular in recent years is a 0.5 m thickness of drainage fill around the landside third of the pipe, regardless of the size and type of pipe to be used (U.S. Army Corps, 2000, Design of Levees).

2.11.12 Pipe Crossings and Pipe Joint Restraints

Transverse Pipe Line Crossings

Transverse pipe crossings (Photograph 25) cannot be avoided in urban and residential areas. Pipe crossings may be required for water, sewer, gas and buried cables for power, telephone and cable television. The most important aspect of transverse piping crossings is to cutoff seepage along the pipeline. Section 2.11.11 provides further details.

Where possible, transverse pipe crossings should be located in the upper zone of the dike, preferably within the freeboard zone.

Some of the basic construction requirements for proposed pipelines that cross through or under a dike fill are as follows:

- Work shall be timed or procedures taken to eliminate any possibility of flooding caused by the pipeline construction;
- Where considerable foundation settlement is likely to occur, camber shall be used to eliminate future sags;
- Seepage collars and/or cutoff walls shall be placed along the length of the pipe;
- The walls of the pipe trench shall be roughened to allow a proper bond between the pipe fill material and the dike;
- Pipe bedding shall be well-compacted, fine-grained material limited in depth to the minimum necessary to create a continuous interface with the pipe;
- Pipe bedding material within the body of the dike fill shall not be more pervious than the existing dike material;
- Compaction of replacement fill shall be to 95% Standard proctor;
- Crest running surface material, if different than the dike fill, shall be replaced to original standards;
- All disturbed grassed slopes shall be replanted;
- All disturbed riprap shall be replaced to original or better condition;
- Wherever the proposed pipeline crosses the dike fill, appropriate metal signage shall be put in place marking the exact centreline location; and
- All valves shall be fully encased.

Longitudinal Pipeline Placement

Installation of longitudinal pipelines within the dike structure shall not be permitted. The dike shall not be viewed as a convenient, inexpensive route for pipelines but as an essential public safety structure whose construction integrity shall not be compromised. Where pipelines parallel a dike near the landward side toe, the alignment shall be beyond the dike structure and seepage layer. The pipeline must not hinder the free movement of maintenance equipment. As well, trench excavation during installation or repairs must ensure continued dike integrity. Warning signs shall be posted at regular intervals along the pipe alignment. Regular inspections shall be carried out to check for subsidence or settlement along the pipeline. Where there is no alternative to installing pipes near the landward side of the dike, ductile iron or steel pipe shall be used. All such pipes are to be kept outside of the typical core of the dike and only permitted where absolutely necessary in an overbuilt fill.

Pipe Joint Restraints

Joint separation can occur due to hydraulic forces and differential settlement including pipe sag. Pipe joints in transverse pipes shall be minimized, with preference given to continuously welded pipes. Where

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Photograph 26: Pipe Jacking

joints exist, adequate restraint must be provided to prevent joint separation.

Joint restraint could involve thrust blocks, tie backs, or reverse camber. Reliance on reverse camber shall only be considered in low pressure situations and where long term settlement can be reliably predicted.

Pipe bends and fittings shall be avoided within the dike. However, where unavoidable, pipe bends and fittings shall be restrained with thrust blocks and anchored. Warning signs shall be posted at regular intervals along the pipe alignment. Regular inspections shall be carried out to check any subsidence or settlement along the pipeline.

Pipe Jacking

Open cut and cover is the preferred method of transverse pipe installation through dikes. Pipe jacking (Photograph 26) is a highly specialized task and expert advice shall be sought if this installation method is pursued. Experienced specialized contractors shall complete the installation.

The pipe jacking procedure generally uses a casing pipe of steel or reinforced concrete that is jacked through the soil. Pumping bentonite or suitable lubricants around the outside of the pipe during the jacking operation often reduces jacking loads. Typically, jacks are oversized so they can be operated at a lower pressure and maintain a reserve jacking capacity.

It is common to install the pipe in a continuous operation to reduce the possibility that the pipe will "set" in the ground as pore water pressures dissipate. Grout plugs in each section of pipe can be used to pump lubricants around the outside of the pipe during the jacking

operation and to pump grout around the outside of the pipe after the push is completed.

If the jacking pit encroaches on the landside toe, a seepage collar can be provided within the dike structure. The jacking pit shall be filled with well-compacted dike fill material.

A drainage detail shall be provided that is adequate to prevent excessive seepage and piping at the landside toe. The detail may consist of buried drainage features with suitable filter, drainage collection and discharge elements, an inverted filter and weight berm above the toe of the dike and the pipe installation pit, or a combination of these.

Directional Drilling

Directional drilling for performing crossings of a wide variety of surface obstructions was developed in the 1970's by combining techniques employed by conventional road boring and the horizontal **directional drilling** of oil and gas wells. This method is now widely employed for the placement of pipelines and conduits for petroleum products, fiber optic and electric cables, water and waste water lines, and for the transmission of other products.

The process generally includes three distinct phases beginning with the drilling of the pilot hole from the surface on one side of the obstacle to be crossed. Drilling continues along a designed profile below and beyond the obstacle to exit at the surface on the other side. The second phase entails the reaming of the pilot hole to a diameter sufficiently large enough to accept the pipeline or conduit. Finally, the pipeline or conduit is pulled into place within the enlarged hole.

Photograph 27: Typical Gate Configuration

The advantages of the **directional drilling** installation method over alternative construction methods are numerous. Environmental issues are minimized as operations are limited to relatively small areas at each end of, as opposed to, the complete length of, the installation.

Normal activity, such as river and highway traffic, can continue unhindered during installation as it is not disrupted along the surface of the installation. Safety concerns associated with trenching or open cutting in rivers or other waterways, or when deeper installations are required on land, are eliminated.

However, particular consideration must be given to soil conditions and access. As with pipe jacking, suitable drainage details may be required once the work is complete. Professional geotechnical advice should be engaged, in particular, depending upon the nature of the crossing.

2.11.13 Gates and Fences

Cross fences shall not impede access along the dike crest for maintenance purposes. Longitudinal fencing shall be adequate to prevent livestock from accessing the dike body and shall not impede the mowing of the dike slopes. Copies of keys to locks must be retained by the responsible dike maintenance and flood response personnel.

Gates (Photograph 27) shall be wide enough to allow the unimpeded access of construction, maintenance and/or inspection vehicles.

Gate locks shall be designed to deter vandalism.

2.12 Design Aspects of Operations and Maintenance

During the planning and design stage, attention shall be paid to the maintenance aspects of the flood protection works. The works shall be designed to provide easy access for regular inspections and more importantly during emergency situations, where materials and equipment may have to be transported in the shortest possible time, to repair washouts and slope failures.

Unlike concrete structures or paved roadways, dikes generally built of earthfill material over weak foundations, are subject to settlement, current erosion, surface erosion due to rainfall runoff and vehicle damage to dike crest and shoulders. Dikes are also subject to damage by burrowing animals, and seepage and piping leading to sloughing and slope failures therefore, regular maintenance is a high priority task.

Consideration shall be given to incorporate access roads, access ramps, turnouts and turn arounds in the original design. Also fences and gates shall be provided to prevent unauthorized access and vandalism. Cost of these provisions shall be considered as part of the project costs of the flood control works.

2.12.1 Operation and Maintenance (O&M) Manual

It is the responsibility of the design engineer to prepare an O & M Manual. This shall be done soon after the commissioning of the works but prior to handing over the project to the diking maintenance authority or the private owner for maintenance.

If a pump station structure is built with the flood protection works, a separate O & M manual shall be prepared for the specific structure.

The IOD can provide the most recent templates used for preparation of operations and maintenance manuals.

Supplementary information such as record drawings, reports, etc. can be appended to the O & M Manual.

2.12.2 Access Roads

Access roads are required for dike repairs, upgrades, and emergency works. Therefore, a permanent right-of-way shall be established for all dike maintenance access roads and, with the exception of security gates, shall be kept clear of obstructions at all times. The minimum roadway width shall be 4.0 m. Minimum road elevations shall be reviewed with the local Diking Authority to ensure the road elevation exceeds the normal internal flood levels. The road surface shall be capped with well-graded crushed gravel or paved.

Access roads connecting the dike to the local road system, shall be maintained even if they go through private properties. Arrangements shall be made with the property owners to provide access for emergency vehicles and crews which may be necessary during high flow periods and winter season. Access shall be available at all times for inspection and maintenance.

The dike crest shall be surveyed not later than two years after the completion of works to check settlement of the crest. Settled sections shall be restored by placing crest gravel, grading and rolling. Thereafter, the dike crest shall be surveyed every five years.

2.12.3 Management of Approved Vegetation

To facilitate inspection of dikes, and future upgrades or repairs, only certain vegetation growth is allowed on dikes (Photograph 28). Trimmed grass is the only vegetation that shall be established on dike slopes. This will reduce surface erosion due to rain, current and waves.

Tree and shrub planting is not recommended on dike slopes. Trees detrimentally affect dike fills by root penetration causing cracking, loosening, wind throw holes and seepage. During windstorms, falling trees can take a large ball of soil from the dike slope leading to erosion and slope failure. Roots in the dike can also cause seepage and piping at high river levels. Trees can also displace riprap bank protection, leading to holes where erosion and instability

may occur. Trees, brush and tall vegetation on dike slopes obstruct the inspector's view to detect seepage, piping and animal burrows.

No new planting of trees or shrubs is to be undertaken without the written approval of the IOD.

The "Environmental Guidelines for Vegetation Management on Flood Protection Works to Protect Public Safety and the Environment (MELP, DFO, 1999) provide direction on acceptable planting and vegetation configurations.

2.12.4 Vandal Proofing

The dike system cannot be made completely "vandal proof." The main objective of vandal protection for dikes is to prevent unauthorized motor vehicles from accessing the dike works and restricting access to pump stations. The following precautionary measures will help to reduce damage by vandals:

- Continuous fencing along the landside of the dike;
- Cross fencing and strong gates installed at access points;
- Tamperproof padlocks on gates;
- Security fencing around pump station structures;
- Heavy duty steel doors on pump station control rooms;
- Security steel grills for ventilation and other openings;
- Steel plate covers with locks for pump chamber openings and access openings; and
- Flood lighting around the pump station yard.

2.12.5 Inspections

It is the responsibility of the diking maintenance authority, or the private owner, to conduct routine and periodic inspections and maintain and repair the works to the same flood protection standard that the works were built to. Necessary funds shall be budgeted

Photograph 28: Vegetation Management

annually and set apart for the specific purpose of maintenance and repair of the flood control works.

The entire flood protection system shall be inspected in detail at least once every year. Such inspections shall be scheduled prior to the high flow season allowing sufficient time to carry out any repair work needed to put the works back to design standards.

Additional inspections may be necessary after each winter season to check slope erosion or sloughing, frost damage or vehicle damage to the dike crest, access ramps, access roads and turn arounds. Annual inspection reports shall be kept and may be asked for by the IOD as part of their audit.

2.12.6 High Water Inspections

Inspections shall be carried out during high water events to monitor the performance of the flood control works.

Special attention shall be paid to check seepage through the dike and functioning of the dike toe drainage filters or concentration and increase in seepage flow through certain areas. Piping and transportation of fine material such as silt and sand in seepage flows on the landside slope or toe area can be noticed by a spring, upwelling sandy silty water along the toe area. Immediate action is required to control such areas, by building sand bag rings around the boil and placing gravel filters. In controlling a boiling spot, the boil may reappear in another area. Sufficient time must be spent to ensure that the boiling is under control.

2.12.7 Burrowing Animals

Burrowing animals are another problem source for earthfill dikes. Bank beavers and rodents burrow through the dike

Photograph 29: Bank Protection on Outlet Side of Dike

creating a system of tunnels. During high water periods seepage and piping can weaken the dike embankment leading to erosion and local failure of the dike section.

It is essential to keep the dike slopes clear of shrubs and tall grass, in order to detect the animal burrows during routine inspections. Trapping of the animals may be one way to reduce this problem.

Once the animal burrows are discovered, the holes and tunnels shall be excavated and backfilled with suitable material and compacted.

2.12.8 Bank Protection Works

Bank protection and erosion protection (Photograph 29) works along the dike and at inlet and outlet areas of floodboxes and pump stations must be regularly inspected and maintained to keep the works up to the original design standards.

For maintenance and repairs, the "Riprap Design and Construction Guide", (MELP, 2000) may be consulted.

3.0 CONSTRUCTION

3.1 Site Preparation

3.1.1 Access

Suitable access from adjoining road or street systems is required.

3.1.2 Clearing, Grubbing and Stripping

For new dike construction, changing the existing alignment or widening of the existing fill, the dike area shall be properly cleared, grubbed and stripped (Photograph 30). Initially the extent of the dike footprint and any additional berms shall be surveyed and marked for the exact boundaries required to do the work (Photograph 31).

The typical extent of clearing, grubbing and stripping is given below:

Clearing
The entire dike right-of-way including the dike footprint, and any proposed berm footprints shall be cleared prior to any further construction operations. Clearing consists of complete removal of all objectionable and/or obstructional matter above the ground surface which includes all trees, fallen timber, brush, vegetation, loose stone, abandoned structures, fencing, and similar debris. All cleared material must be disposed of by approved means.

Grubbing
The entire dike footprint shall be grubbed following clearing operations. Grubbing is usually not necessary beneath additional berms. Grubbing consists of the removal of all stumps, roots, buried logs, old piling, old paving, drains, and other objectionable matter. Roots or other intrusions over 50 mm in diameter within the dike foundation area shall be removed to a depth of 1 m below natural ground surface. All holes and depressions caused by grubbing shall be flattened and then backfilled and well-compacted to avoid "soft spots" under the dike and maintain the continuity of the natural blanket. All grubbed material must be disposed of by approved means.

Stripping
The entire dike footprint, and any existing dike surfaces which require additional materials, shall be stripped following foundation clearing and grubbing. Stripping is usually not necessary beneath additional berms. Stripping consists of the removal of low growing vegetation and organic topsoil. The depth of stripping is determined by local conditions and typically varies from 150 to 300 mm. In preparation of new dike fills, the ground shall be scarified following stripping. All stripped material suitable for use as topsoil shall be stockpiled for later use on the slopes of the embankment and berms. Unsuitable material must be disposed of by approved means.

Photograph 30: Site Preparation Prior to Dike Upgrade

3.1.3 Dewatering

Areas where minor seepage inflow is expected during subexcavation of materials or trench excavation can likely be treated using conventional ditching and sumping techniques which are relatively inexpensive. Areas with moderate to major seepage will likely require more extensive and costly dewatering methods such as well point dewatering.

If settlements around the construction site are expected due to dewatering, consideration may be given to groundwater control methods such as recharge wells, infiltration ditches, or other approved alternatives to avoid lowering of water levels beyond the construction site.

To avoid expensive dewatering, consideration may be given to designing the dike with seepage control measures situated above the water table (such as berms), or some degree of underseepage may need to be accepted. If significant dewatering is expected to be required during placement and compaction of fills due to subexcavation from stripping, consideration may be given to using fill materials that can be more readily compacted under saturated conditions.

3.2 Construction

3.2.1 Excavation

Stable excavation below the existing ground surface is highly dependant on foundation soil and groundwater conditions. Any proposed excavations shall be reviewed by a qualified professional geotechnical engineer registered in B.C. during the design and construction stages of a project.

If space permits, the excavations can typically be carried out with open cut slopes, together with the installation of suitable dewatering measures. For most soil conditions with limited seepage, unsupported excavation side slopes can typically be developed at 1.5 Horizontal to 1 Vertical (1.5H:1V) or flatter. These cuts shall be flattened to 2.5H:1V if significant seepage and/or sloughing occurs or alternatively, prepared as described below.

For unsupported excavations greater than 3 m deep or carried out below the groundwater level, large, open cut excavations may be impractical. Even with use of flat slopes, significant sloughing may occur (depending on soil conditions). Consideration can be given to carrying out excavations using suitable temporary excavation support and dewatering to maintain a stable, dry excavation. Such support can be provided using a moveable trench box or using suitably braced shoring. The contractor shall engage a qualified professional geotechnical engineer registered in B.C. to design and inspect all temporary works proposed and to ensure safety and compliance with WCB Regulations. The temporary shoring shall be braced to resist the anticipated soil loads and differential hydrostatic loads, including surcharge loads resulting from construction equipment and/or any material stockpiles. Measures shall be taken to minimize the risk of base heave when using open cut or shored systems. The method of excavation shall be reviewed on a case by case basis, considering the conditions at the time of construction.

3.2.2 Excavation adjacent to or through existing works

Excavations adjacent to or through

Photograph 31: Survey Layout For Dike Upgrading

Photograph 32: Vibratory Roller

existing dikes, bank protection or other flood protection structures, and excavation near existing structures shall be avoided, but where necessary, expert advice shall be obtained to ensure that the excavation is compatible with stability of the works.

If excavation is necessary, suitable support to any critical utilities and structures will need to be provided to minimize the potential vertical and/or lateral deformation of the critical structures associated with the construction operations. Typically, any excavation slopes shall be set-back at least 1 m horizontally from the existing critical structures and the slopes cut at 1.5H:1V, or flatter under dewatered conditions and subject to review by a qualified geotechnical engineer. Alternatively, suitably designed shored systems shall be used to prevent ground movements and deformation of the existing critical structures. If it is considered acceptable for some of the critical structures (such as utilities) to be repaired if broken, shoring systems may not be required.

It is also recommended that monitoring gauges be installed on the existing critical structures to permit measurement of any vertical and lateral deformations. Gauges shall be monitored prior to, during, and after construction which is located close to any critical structures. The readings shall be forwarded to a qualified geotechnical engineer for review and analysis.

In many cases drainage ditches are proposed near dikes. Ditches close to the landside toe of the dike can lead to both seepage and/or slope stability problems. The location, depth, and feasibility of any proposed ditch shall be

Photograph 33: Sheepsfoot Roller

determined by seepage and stability analyses.

3.2.3 Fill Placement/Compaction

Compaction Fundamentals

Soils containing fines can be compacted to a specific maximum dry density with a given amount of energy; however, maximum density can be achieved only at a unique water content called the optimum water content. Maximum dry density and optimum water content are determined in the laboratory by carrying out Proctor testing on collected samples.

Compactive effort can be increased by increasing contact pressure of the roller on the soil, increasing the number of passes, or decreasing the lift thickness. Combinations of these procedures to increase and control compaction on a job will depend on difficulty of compaction, degree of compaction required, and economic factors (Photograph 32).

Compaction of Dike Fills

Requirements of the more important compaction features, such as water content limits, layer thickness, compaction equipment, and number of passes will be contained in the specifications and must be checked closely by the inspector to ensure compliance. Specifications will generally state the type and size of compaction equipment to be used.

Uncompacted or loose lift thickness will be specified. Lift thickness specified will be based on type of material and compacting equipment used. Impervious or semipervious materials are commonly placed in 150 to 200 mm loose lift thicknesses and compacted with six to eight passes of a sheepsfoot roller, or an approved alternative (Photograph 33).

When using any roller that leaves a smooth surface after compaction, scarification of the compacted lift prior to placing the next lift is specified to ensure a good bond between the lifts. Pervious materials, less than about 10 per cent fines, are commonly placed in 300 mm loose lift thicknesses and compacted with four to five passes of a vibratory steel-wheel rollers in the weight range of 5 to 15 tons, or an approved alternative.

In-place water content and density must be related to optimum water content and to maximum dry density to judge whether a compacted soil is suitable or unsuitable. Minimum acceptable field density is normally established in design as a percent of maximum dry density, and an allowable range of placement water contents is given in the specifications relative to optimum water content of the soil being compacted (Photograph 34). Commonly, the specification calls for a minimum 95 percent of the Standard Proctor Maximum Dry Density (ASTM D698, Method C). Each soil type has a different maximum dry density and optimum water content for a given compactive effort, and it is necessary that in-place field densities and water contents be compared with laboratory-determined optimum water contents and maximum densities of the same soil. Because mixing different soil strata in borrow areas can result in materials with unexpected compaction characteristics, if a material being compacted in the field cannot be related to available laboratory compaction data, a laboratory compaction test shall be performed on that material. Check companion tests shall be performed by field personnel before fill placement to ensure

Photograph 34: Water Placement For Optimum Soil Moisture

consistency with target values for a given soil.

Compaction of Drainage Layers

Placement and compaction of drainage layers must ensure that adequate density is attained, but shall not allow segregation and contamination to occur. Vibratory rollers are probably the best type of equipment for compaction of cohesionless material although crawler tractors and rubber-tired rollers have also been used successfully. Saturation or flooding of the material as the roller passes over it will aid in the compaction process and in some cases is the only way specified densities can be attained. Care must always be taken to not overcompact to prevent breakdown of materials or lowering of expected permeabilities. Loading, dumping, and spreading operations shall be observed to ensure that segregation does not occur. Gradation tests shall be run both before and after compaction to ensure that the material meets specifications and does not contain too many fines.

Rapid Impact Compaction

Rapid impact compacting (Photograph 35) involves the use of a high speed compacting unit that maintains continued contact with the soil surface, to improve the engineering properties of a wide range of fills. It has demonstrated improvements in miscellaneous fills. Advantages include:

- Speed of operation;
- Lack of introduction of other

Photograph 35: Rapid Impact Compaction

materials such as water; and
- No time lapse before the fills can be built upon.

Compaction Grouting

Compaction grouting is the injection of a viscous soil-cement grout under pressure into the soil mass, which consolidates and densifies targeted soils insitu.

When injected into poorly compacted soils, grout pushes the material aside and occupies void space, thereby improving compaction of displaced soils and providing a more uniform soil mass. Applications include densification of foundation soils and mitigation of liquification potential. The principal application of this technique is densification of soils subject to long-term settlement.

Reinstated Dike Fills Abutting New Concrete Structures

Where there is a possibility of seepage due to a reinstated dike fill abutting a new concrete structure, the following techniques have proven successful to ensure adequate protection against seepage.

Seepage plates (or barriers) shall be installed where disturbed dike fill will abut new, smooth concrete walls. These barriers shall be placed at the centre of the dike crest to the depth of dike fill disturbance and shall extend outwards by 0.6 metres. Barriers could either be metal, affixed securely and watertight to the abutment wall, or constructed out of concrete, extending 0.6 metres outward with a minimum of 0.15 metres thickness, provided proper bonding to the abutment can be achieved.

Embankment Construction Deficiencies

Typical construction deficiencies include the following:

- Organic material not stripped from foundation:
  - Differential settlements; shear failure; internal erosion caused by through seepage
- Highly organic or excessively wet or dry fill:
  - Settlements; inadequate strength
- Placement of pervious layers extending completely through the embankment:
  - Allows unimpeded seepage which may lead to internal erosion and failure of the embankment
- Inadequate compaction of embankment, such as lifts too thick:
  - Settlements; inadequate strength through haphazard coverage by compacting equipment, seepage
- Inadequate compaction of backfill around structures in embankment:
  - Settlements; inadequate strength; provides seepage path between structure and material which may lead to internal erosion and failure by piping

3.2.4 Riprap and Filter Layer Placement

Riprap and filter layer placement is best described in the publication "Riprap Design and Construction Guide – March 2000" available from the office of the IOD. The following procedures however provide an overview of the requirements of riprap placement.

- Rock for use as riprap must be placed in such a manner as to produce a reasonably well-graded mass of rock with the minimum practicable percentage of voids;
- Rock for use as riprap must be constructed within the specified tolerance to the lines and grades shown on the design cross-sections;
- No rock shall protrude more than 0.3 metres above the lines and grades shown on the design cross-sections;
- Rock riprap material shall not be dropped from a height greater than three (3) metres vertically from its final position;
- The larger pieces of rock shall be well distributed and the entire mass of rock in its final position shall be roughly graded to conform to the gradation as specified in the design; and
- The finished riprap shall be free of objectionable pockets of small stones and/or clusters of larger stones.

Placing riprap by end dumping from haul units over the bank or similar methods likely to cause segregation of the various sizes in their final position will not be permitted. This placement technique can also result in breakage of riprap units.

The upstream end of the riprap must be keyed into the bank or physically connected to a relatively permanent structure such as a solid rock outcrop to help prevent outflanking. The downstream end of the riprap must be of such an alignment and configuration that it does not cause a scour hole to develop.

3.2.5 Final Grading

A survey of the dike crest is usually needed for the final grading to ensure there are no low areas. Any low spots along the crest of the dike are to be filled.

The dike crest shall be graded with a slight camber to prevent water from sitting on the crest. If the dike crest running surface is of a pervious material, it shall be placed above the design crest level. The running surface usually is constructed above the freeboard.

Sideslopes are to be trimmed, usually with a hydraulic excavator with a cleanout bucket or a Gradall, to the required slope. Topsoil can be placed onto the side slopes that require a medium to promote the growth of grass, and the sideslopes can then be hydroseeded. The vegetation will serve to protect the slopes from erosion by surface runoff water.

3.2.6 Timing/Construction Staging

The issue of construction timing and staging is of importance with respect to water levels, aquatic habitat and weather. **DFO has clearly noted schedules when instream works are to be undertaken, thus it is important to review and confirm these schedules prior to tendering.**

Construction timing can also be water level driven. Works in and about streams shall be undertaken during low flow periods in order to take advantage of a broader exposure of bank and or shoreline. This can facilitate ease of construction and decrease costs.

Construction timing is also driven by tide levels. Daytime low tides, for example, in late August and early September, provide a suitable work window for shoreline works on tidewater. At other times of the year, daytime low tides can be higher, thereby limiting above water work.

3.3 Monitoring

3.3.1 Site Inspection – Construction Monitoring

Simple controls using both visual observations and rough measurements are the primary means by which construction monitoring is carried out. However, they must not be used as the only means of monitoring, but must be supplemented by a program of control testing. For any estimate to be meaningful and accurate, the observer must have his eye and hand calibrated to all conditions expected. It is desirable to construct a small test section prior to the beginning of major fill placement so inspectors and the contractor can become familiar with the behavior and compaction characteristics of the fill material and with the performance of the compacting equipment.

Gradation tests shall be performed to ensure that the material being placed is within specification limits. The number of gradation tests needed will depend on the variability of material as obtained from the borrow areas. Complete gradation tests shall be performed on material for which the entire range of particle sizes is specified.

Proper lift thickness is fairly easy to estimate when the inspector's judgment has been calibrated by actual thickness measurements. However, many contractors are interested in placing lifts as thick as they can get by with, and conflict often arises on this point. Therefore, control of lift thickness by visual observation alone is not sufficient and must be supplemented with measurements.

Field control testing, field density tests and record sampling of compacted fill are conducted for two basic reasons: to ensure compliance with design requirements, and to furnish a permanent record of as-built conditions of the embankment. Field control testing consists largely of determinations of the water content, density, and classification of the field-compacted material. Record

sampling consists of obtaining undisturbed samples, often with companion disturbed bag samples at selected locations in the embankment during construction.

Frequent density testing shall be carried out at the start of fill placement; after rolling requirements have been firmly established and inspection personnel have become familiar with material behavior and acceptable compaction procedures, the amount of testing can be reduced. Sampling shall be carried out at locations representative of the area being checked. A systematic testing and sampling plan shall be established at the beginning of the job. Control tests are usually designated as routine and are performed at designated locations, no matter how smoothly the compaction operations are being accomplished. A routine control test shall be performed for every 1000 to 2500 m^3 of compacted material and even more frequently in narrow embankment sections where only a small volume of material raises the section height considerably. In the first lift above the foundation, tests shall be made more frequently to ensure that proper construction is attained in this important area. The locations of record samples shall be at the discretion of the design engineer and shall also be stated on a predetermined plan of testing.

In addition to routine density tests, tests shall be made in the following areas: where the inspector has reason to doubt the adequacy of the compaction, where the Contractor is concentrating fill operations over relatively small areas, where special compaction procedures are being used (power tampers in confined areas, etc.), where instruments are located, and adjacent to structures.

Currently, the nuclear method is a common means by which both water content and density determinations can be made more rapidly than by conventional direct methods, which were commonly used in the past. Most nuclear gauges are built to measure density by one or more methods, classified as the direct transmission, backscatter, and air-gap density methods; however, all nuclear gauge methods are based on the principle of using gamma radiation to establish a density relationship.

The advantage of the nuclear method is the speed with which density and water content determinations can be obtained as compared with conventional methods. However, the field density and water content must still be related to a compaction curve or to maximum and minimum densities, as is the case with data obtained by conventional methods. Consequently, it is necessary to occasionally obtain samples of the material at the location of the nuclear test in order to relate the field and laboratory data.

3.3.2 Environmental Monitor

An Environmental Monitor is often required and allowances for such services shall be included in the design estimates. The Inspector onsite can also fulfill the role of Environmental Monitor if they are deemed suitably qualified by

the approving environmental agencies. Generally, the Environmental Monitor will have authority to stop work for work practices that are not in compliance with the environmental permits for the job, which is similar to the role of the Inspector.

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APPENDIX I

GLOSSARY

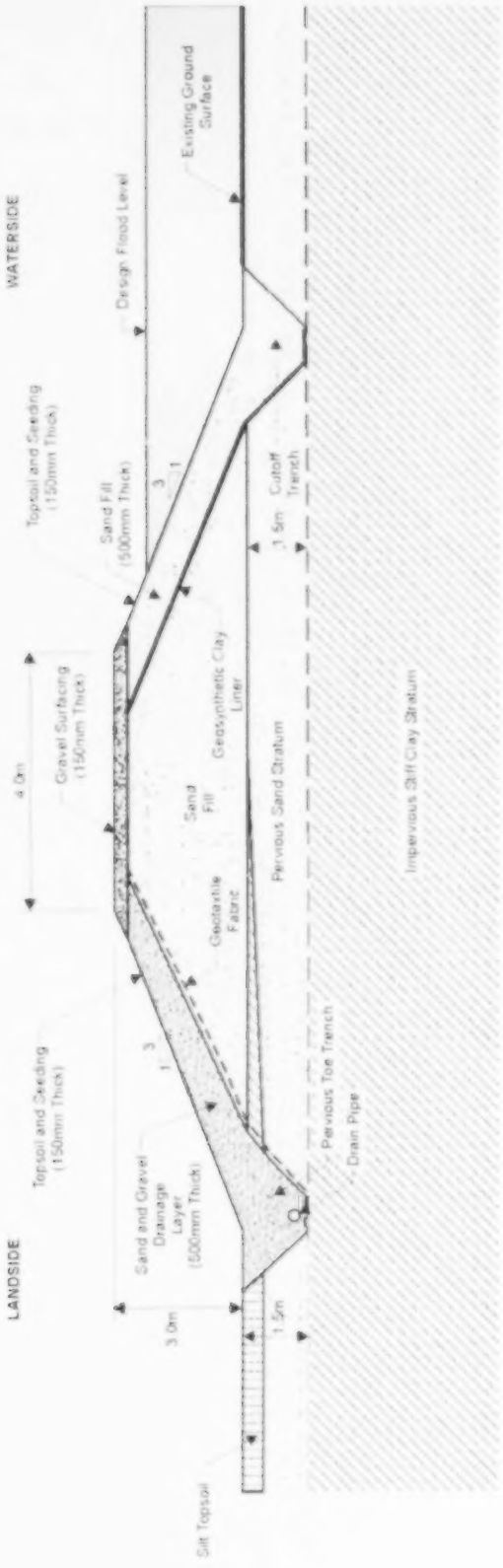
Alluvial Fan:	Fan shaped deposits of predominantly water borne materials.
Apex:	Upper limit of depositional area of an alluvial fan.
Bank Protection:	Treatment of slopes of dikes and banks of streams, lakes and other water bodies by placement of riprap (an engineered layer of graded broken rock pieces) or other forms of protection to prevent erosion by surface runoff, stream flows and/or wave action.
DIOD:	Deputy Inspector of Dikes.
Debris Flood:	A very rapid, surging flow of water, heavily charged with debris, in a steep channel.
Debris Flow:	Very rapid to extremely rapid flow of saturated nonplastic debris in a steep channel.
Dike:	An embankment, berm, wall, piling or fill constructed to control flooding of land.
Dike Height:	The vertical distance from the dike crest level to natural ground as measured at the landside toe of a dike.
Erosion:	Loss of land or bed materials due to action of flowing water which can be regular or highly episodic.
Excessive Vegetation:	Growth such as blackberry and salmonberry whose pervasive presence obscures visibility and inhibits access.
Flash Flooding:	Rapid rise in creek or river levels due to intense rainstorms or rain or snow events.
Freeboard:	Vertical allowance added to standard design flood level to allow for uncertainties in flood levels.
Freshet Flooding:	Spring snowmelt runoff influenced by annual winter accumulation of snowpack and specific temperature and rainfall conditions in the spring.
Global Warming:	Long term rise in mean atmospheric temperature due to greenhouse effects.

IOD:	Inspector of Dikes.
Ice Jam:	Stationary accumulation of ice that restricts flow.
Overbank:	The area of land between the waterside toe of a setback dike and the top of the streambank.
Overwidth Dike:	A dike having standard dike side-slopes (or flatter) and a minimum 9 metre crest width measured from the landside crest edge. Overwidth dikes are sometimes formed by roads or dikes constructed beside natural riverside levees.
Natural Riverbank:	The bank of the river, formed naturally and not part of the dike fill; located below the dike height on the river side.
Piping:	Internal erosion of embankment materials due to seepage.
Right-of-Way:	A legally defined strip of land to provide access for maintenance.
Riparian Vegetation:	The vegetation immediately in contact with a water body or sufficiently close to have direct influence on aquatic habitat values.
Riverside Dike:	A dike located adjacent to a stream, directly on a streambank. Riverside dikes may be constructed with or without bank protection.
Seepage:	Movement of water through soil under differential head.
Setback Dike:	A dike that is set back from the ordinary high water mark of a river creating an overbank strip of natural ground between the dike fill and the riverbank.
Sliver Fill:	A narrow fill wedge occurring due to widening of an existing sloped surface.
Standard Design Flood:	A flood with an annual probability of occurrence of 0.5%, also known as a 1 in 200 year flood.
Storm Surge:	Rise in mean water level due to barometric or atmospheric effects.

Tides:	The regular rise and fall of ocean water levels due to the gravitational influence of the sun and moon.
Tsunamis:	Seismically or landslide generated waves.

APPENDIX II

SAMPLE DESIGNS
(FOR DEMONSTRATION PURPOSES ONLY)



Design Example: Seepage Control

Design Example 1: Seepage Control

Problem:

- To design an acceptably stable dike (with limited seepage through and beneath the dike) to contain a design flood event and associated forces with a 200 year recurrence period interval (1:200 year), and an appropriate additional freeboard allowance.

Known Design Criteria:

- The required height of the dike above the existing ground surface, based on a 1:200 year flood event, is 3 m. This includes a 0.6 m freeboard allowance.
- A 4 m minimum crest width allowance for maintenance vehicles, with a minimum 150 mm thick gravel surfacing.
- A minimum 150 mm thickness of topsoil and subsequent hydroseeding on the finished sideslopes.
- Winter construction period with periods of heavy precipitation expected.

Soil and Groundwater Conditions:

- The soil and groundwater conditions at the site were determined by auger drilling investigation along the dike alignment. Dynamic Cone Penetration tests (DCPT's) were carried out adjacent to some of the auger holes to determine the relative density of the soils along the proposed dike alignment. Standpipe piezometers were installed in some of the open augerholes to allow continued monitoring of the groundwater level along the alignment.
- The landside portion of the proposed dike alignment is underlain by a thin layer of silty topsoil (less than 300 mm thickness), which tapers out towards the waterside of the dike.
- A pervious sand stratum of up to 1.5 m thickness exists at the surface of the waterside of the proposed dike and below the topsoil layer on the landside of the proposed dike. Based on the DCPT results, this layer is inferred to be relatively compact.
- A relatively impermeable stiff clay stratum underlies the pervious sand stratum.
- Based on the readings collected from the standpipe piezometers, the groundwater table is expected to be between 0.5 and 1 m depth below the existing ground surface at the time of construction.
- A borrow pit comprising generally sand with a trace of gravel and about 5 to 10 per cent fines (silt and clay sizes), exists near the site and the material is economically available. This material will be utilized wherever possible in the final design.

Geotechnical and Hydrological Analyses:

- Detailed slope stability and seepage analyses was initially carried out assuming only the available sand (well compacted) is used to construct the dike to the required 3 m height above the existing ground surface.
    - Based on our combined seepage and stability analyses, excessive seepage is expected both through and beneath the dike during a design flood event. In addition, even with relatively flat 4 Horizontal:1 Vertical (4H:1V) dike sideslopes, piping, erosion, and sloughing may occur, and there is a possibility of a more extensive failure through the dike. Based on these analyses, and assuming the available sand is to be used wherever possible, seepage control and dike zoning will be required to achieve an acceptable level of stability and seepage control during a design flood event.
- Based on our subsequent detailed slope stability and seepage analyses, the following dike design limits seepage through and beneath the dike and achieves an acceptable level of stability during a design flood event. This design does not consider the effects of seismic loading or possible liquefaction potential (due to seismic loading) at the site. It should be noted that a number of other designs are possible to control seepage and to achieve an acceptable level of stability during a design flood event with the same design criteria:
    - A sand and gravel drainage layer and pervious toe trench are proposed at the landside of the dike. The pervious drainage layer will provide an exit for seepage through the embankment and can lower the phreatic surface sufficiently so that no seepage will emerge on the landside slope. This drainage layer is combined with a pervious toe trench as a method for controlling shallow underseepage by reducing the potential for a buildup of seepage pressure (due to the existence of the thin topsoil layer). A geotextile fabric is proposed between the pervious drainage layer and the bulk dike fills (the available sand), to provide a separation/filter layer to prevent the finer sand fill from migrating into the coarser sand and gravel fill.
    - Dike sideslopes of 3H:1V are proposed. Based on our analyses, the sideslopes will have an acceptable level of stability during a design flood event. In addition, they are flat enough for future mowing of the slopes.
    - Although the stability of the dike is acceptable using the above methodology, the expected level of seepage both through and under the dike would still be higher than desired due to the generally pervious nature of the dike fills and subsurface soils. To reduce seepage, and to further improve stability, an impervious layer and cutoff trench are proposed at the waterside of the dike to greatly reduce seepage through and beneath the dike. Due to the possibility of heavy precipitation during construction, a relatively impermeable silt and clay fill material is not considered to be feasible due to the expected difficulty to compact and transport these

materials during periods of heavy precipitation. A geosynthetic clay liner is proposed as shown, with the liner covered along the waterside sideslope and the cutoff trench filled with the available sand fill material. This liner should be adequately anchored at the top of the dike to prevent it from sliding. The cutoff trench would be excavated to and keyed into the impervious stiff clay stratum which underlies the site. It should be noted that such a liner would need to be installed based on the manufacturers specifications and care would need to be taken to avoid tearing this material during construction. Some additional seepage should be expected in areas where tears in the liner occur.

Additional Comments:

- It should be noted that this is a design example provided for discussion purposes only to provide some methods used successfully in the past and for consideration in future designs. Actual site specific dike design and analyses should be carried out by qualified professionals and dike construction should be carried out with an acceptable level of quality assurance including site inspections to confirm that the specified dike criteria are being met.

Design Example 2: Soft Ground Conditions

Design Example 2: Soft Ground Conditions

Problem:

- To design an acceptably stable dike (with limited seepage through and beneath the dike) to contain a design flood event and associated forces with a 200 year recurrence period interval (1:200 year), and an appropriate additional freeboard allowance.

Known Design Criteria:

- The required height of the dike above the existing ground surface, based on a 1:200 year flood event, is 2 m. This includes a 0.6 m freeboard allowance.
- A 4 m minimum crest width allowance for maintenance vehicles, with a minimum 150 mm thick gravel surfacing.
- A minimum 150 mm thickness of topsoil and subsequent hydroseeding on the finished sideslopes.
- An important utility exists about 50 m from the centerline of the dike (landside).

Soil and Groundwater Conditions:

- The soil and groundwater conditions at the site were determined by auger drilling investigation along the dike alignment. Standpipe piezometers were installed in some of the open augerholes to allow continued monitoring of the groundwater level along the alignment. Testing was carried out on collected soil samples to determine the compressibility and strength characteristics of the existing site soils.
- A highly compressible very soft peat deposit of up to 2 m thickness underlies the proposed dike alignment.
- A moderately compressible soft silt clay stratum underlies the surficial peat deposit.
- Based on the readings collected from the standpipe piezometers, the groundwater table is expected to be between 0 to 0.5 m depth below the existing ground surface at the time of construction.
- A borrow pit comprising generally sand and gravel fill and about 5 to 10 per cent fines (silt and clay sizes), exists near the site and the material is readily available. This material will be utilized wherever possible in the final design.

Geotechnical and Hydrological Analyses:

- The natural peat, silt and clay soils that underlie the proposed dike alignment are highly sensitive to disturbance and considered moderately to highly compressible. As a consequence, any increased loading, such as that resulting from the proposed dike fills, will cause significant long-term consolidation settlement of these sediments as well as give rise to potential stability concerns if the dike is not constructed in carefully controlled stages. In addition, significant lateral soil

movements are also possible within these soft and generally weak soils during construction of the dike.

- Detailed slope stability and settlement analyses were carried out for the project with the following dike design achieving an acceptable level of stability during construction and subsequently during a design flood event and to reduce the potential long term settlements at the site. This design does not consider the effects of seismic loading at the site. It should be noted that a number of designs are possible to stabilize the underlying soils and reduce potential long term settlements with the same design criteria:
    - Installation of instrumentation to monitor the porewater pressure performance within the fine-grained and organic subgrade soils, and ground movement effects;
    - Slope Inclinometers installed about 20 m from the landside toe of the dike to monitor lateral movements adjacent to the dike (side of existing utility);
    - If limited time was available to build the dike, wick drains could be installed within the subgrade soils to facilitate the dissipation of excess porewater pressure and accelerate consolidation of the soils beneath the dike area. This effectively strengthens the subgrade soils and improves the stabilizing effect of the dike, allowing for more rapid dike construction;
    - Base reinforcement such as geogrid and/or geotextile may be installed beneath the dike to improve base stability.
    - The readily available sand and gravel is constructed in stages (maximum 0.5 m thick), with a delay imposed between fill stages to allow for dissipation of excess porewater pressures and strength gain within the underlying soft foundation soils. If excessive porewater pressures buildup too high they can lead to rapid failure of soft foundation soils. Each stage of filling commences when monitoring of instrumentation and engineering analyses confirm that it is safe to do so. Filling continues until some 3 to 4 m thickness of fills are in place and the top of the dike is about 300 mm above the proposed final elevation to allow for future settlements.
    - A final crest width of 5 m is proposed with 300 mm of gravel surfacing. This will provide enough room to "top-up" the dike materials in the future following expected ongoing settlements, and still maintain a reasonable crest width and running surface for maintenance vehicles without the addition of fill materials to the sideslopes of the dike.
    - An impervious layer is proposed at the waterside of the dike to greatly reduce seepage through the dike. An imported silt and clay fill with between about 20 and 40 per cent fines is proposed for this layer. A geotextile fabric is proposed between the relatively impervious silt and clay fill layer and the bulk dike fills (the available sand and gravel fill), to

provide a separation layer to prevent the finer silt and clay fill from migrating into the coarser sand and gravel fill.

- Dike sideslopes of 3H:1V are proposed. Based on our analyses, the sideslopes will have an acceptable level of stability during construction and subsequently during a design flood event. In addition, they are flat enough for future mowing of the slopes.

Additional Comments:

- If a 3 m dike was required instead of the 2 m high dike discussed above, flatter sideslopes and additional construction time would likely be required to maintain adequate stability of the structure. Also, additional settlements would be expected due to the larger footprint and additional weight of materials required.
- Fill placement on peat results in extensive and unavoidable vertical ground settlements, as well as significant lateral movements. In many cases, ground displacements in the order of a few metres can result from the loading of peat sites utilizing carefully controlled staged loading together with regular monitoring. Even larger soil displacements and ground failures (including up thrusting of adjacent land) are common where such loading is undertaken without adequate geotechnical investigation design, construction inspection, and ground movement monitoring to control the rate of fill application. Ground failures can lead to severe damage of adjacent structures or utilities. Once an area has failed, the soft peat and silts reduce to a residual soil strength. It can take months or possibly years for the soils to return to their natural strengths and continued construction would be very difficult during this period.
- It should be noted that this is a design example provided for discussion purposes only to provide some methods used successfully in the past and for consideration in future designs. Actual site specific dike design and analyses should be carried out by qualified professionals and dike construction should be carried out with an acceptable level of quality assurance including site inspections to confirm that the specified dike criteria are being met.